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RED RIVER VALLEY WATER PROBLEMS

By W. P. Tarbell
(*City Engineer, Fargo, N. D.*)

In 1880, which we will accept as the starting point as a background for this problem and about which time government surveys were made, we find over five thousand bodies of water of sufficient size to be classed as lakes. At present there may be one thousand sloughs. Devils Lake which originally had an area of 125 square miles and a depth of 35 feet now has an area of less than 30 square miles and a depth of less than 7 feet. Minnesota lakes which drain into the Ottertail and thence into the Red River have receded, without exception, from two to ten feet. Among these lakes are Ottertail, Rush, Big Pine, Little Pine, Height of Land, Round, Many Point, Little Bemidji, Flat and Egg Lakes. The Bois de Sioux river which joins the Ottertail at Breckenridge, Minnesota, and which formerly carried a considerable flow of water throughout the year, is now dry except during a short period of the spring run-off. The Sheyenne River in North Dakota which empties into the Red below Fargo has in late years been known to be only a series of puddles. This river, which we will refer to again, rises in the central western part of the state, approximately due north of Bismarck, the state capitol. The Wild Rice river which feeds the Red River above Fargo has, for the past few years, been entirely dry except during short periods of spring run-off. The Red Lake River rising in Minnesota and flowing into the Red River at Grand Forks and supplying Grand Forks and East Grand Forks with water has been

very unsatisfactory of late years. The Red River of the North, which is a source of supply for at least ten large communities was classed as a navigable stream until after 1915. Steamboats operated on it and it was the chief means of transportation in the early days. The Federal Government maintained a dredge for navigation purposes until 1915, but there was no flow whatever in the river for 37 days in 1932 and for 156 days so far in 1933.

WELL SUPPLIES

Artesian wells were plentiful in the early days and water was obtainable in any location within a few feet of the surface. At present, wells over the entire area under consideration are failing, some of them completely. In 1931, the City of Fargo made a survey of water conditions within a radius of fifteen miles on the North Dakota and Minnesota sides of the Red River. Very few wells were adequate. The majority of the farms had two, three and four wells, the older ones being dry or nearly so. This survey was made in conjunction with one of the largest well drilling companies in the world and was for the purpose of determining the possibility of Fargo obtaining a supplementary ground water supply. Even wells in the so-called Dakota sandstone strata at depths from over 200 to 500 feet were failing. The Dakota sandstone water is quite mineralized and in some cases quite salty.

The City of Fargo, previous to 1910, obtained its domestic supply from semi-artesian wells in and around the city. About that time these wells began to fail and the City constructed a purification plant. In the past ten years, the City has drilled two wells to granite. Private companies have drilled several others and with nothing at all satisfactory in the way of supply. In 1932 the County drilled two 6-inch wells to granite north of the city which gave a combined pumpage of one and one-half gallons a minute.

The City of Moorhead, Minnesota, in twenty years has gone from one well to seven wells. Its population has not increased more than ten percent and its pumping level has receded 140 feet. Construction of a well by a creamery in Moorhead immediately dropped their pumping level 23 feet.

Wahpeton, North Dakota, across the river from Breckenridge, Minnesota, depends upon the Red River for its supply. As a supplementary supply it has two wells with water of very poor quality which are capable of delivering probably 75 gallons a minute. Breck-

enridge is attempting to obtain a supplementary supply and has expended $7,000.00 to date on their well without satisfaction.

Jamestown, North Dakota has five wells in the underflow of the James River with a very poor quality of water. The James River, incidentally, has been practically dry at times the last few seasons.

Valley City on the Sheyenne River has constructed a dam to impound water and divert this water through a gravel pit into wells. Wells proper have been abandoned in Valley City as inadequate.

The city of Devils Lake, North Dakota, depends for its supply upon two wells 1500 feet deep which deliver water containing 30,000 parts per million of mineral. The water is not potable. For their domestic water they depend upon several shallow wells of limited capacity; the water being delivered from house to house.

Minot, North Dakota, on the Mouse River depends upon wells for its supply. They have constructed a dam to impound water at times when the river is dry. They have three wells of poor quality and are being forced to construct another.

At Grand Forks, North Dakota, the flow of two rivers is available and while adequate is very undesirable due to polution. It is known that in the winter of 1932–33 for several weeks the flow in the river was 5 cubic feet per second, while the total demand was 6.5 cubic feet per second. It is not difficult to figure the percentage of re-circulated sewage which they were required to treat in their plant.

At Fargo, the city constructed a dam in 1910 to impound water for domestic and industrial supply. This dam was raised to the maximum height possible in 1930. Continued diminishing flow in the river over several years indicated the necessity of increased storage. In the winter of 1932, $20,000.00 was expended for an additional dam up stream, impounding the maximum amount of water at that point. Notwithstanding the precautions taken, water reserves have been at times alarmingly low. Flow in the river has forced the city to resort to its reserves frequently.

Referring again to Breckenridge and Wahpeton, flow ceased in the Red River in December, 1932, and their entire reserve storage was consumed. There was no water in their distribution system. Not a bath was taken for ten days, indicating other shortages. Only heroic measures on the part of the two cities and a public utility obtained water for the balance of the winter. Ottertail lake, which is the lowest natural storage at the head waters of the Red River was robbed of all the water which could be released. The above

points will indicate to you the progressively diminishing water supply over a tremendous area with no prospects of permanent relief other than that which will be outlined later.

CAUSES OF DEPLETION

You may be interested in knowing the probable causes of the depreciation of water supply in this area. We have, of course, the apparent wet and dry cycles. Whether they are caused by sun spots or otherwise does not make a great deal of difference. Communities and civilization cannot be entirely dependent upon the natural variation for permanent existence and development. We know that drainage such as has been constructed robs the ground of its storage and decreases or destroys springs and the underground feed of rivers. We know that agriculture has a great effect upon water supply. Twenty bushels to the acre crop of wheat will require five times the moisture that the same area of hay or grass will. Rain which falls upon cultivated land is either drained away artificially or by the time most of it reaches the root stems is returned to the atmosphere through transpiration. A study of the records of flow in the Red River of the North shows that lumbering operations frequently greatly affected the river flow. It was the custom on the watersheds of the lakes mentioned earlier for the lumbering companies to impound practically the entire spring run-off. This water was used to float logs down to the mills. This operation was conducted three or four times a year with the result that the lower lakes were kept full and the swamps and bogs saturated giving an adequate flow in the river. These operations ceased in 1916 and discharge records indicate consistently lower flows each year since. Another point which might be of interest in this problem is that while we have a precipitation average of 17 inches, the evaporation will amount to 33 inches and during the period in which we receive a large portion of our precipitation, the ground is frozen and cannot absorb it.

The results of the depreciation in water supplies in the area under consideration, namely the eastern parts of North and South Dakota and the whole of the Red River Valley, are very apparent. Windbreaks around farm buildings, trees and forests along the rivers and in the lake region are dying off. Scarcely a group of trees remain that does not have many with the tops dead and brown. In agriculture the records of production indicate crops to be unreliable and very reduced in quantity. Authorities in agriculture estimate

thousands of acres of top soil are lost each year through dust storms which have been so prevalent the past few years.

In the matter of domestic supply, many communities and farms have at times had to resort to hauling of water. The most apparent and definite result is the beginning of what appears to be an exodus of people and industries from this territory. To a great extent the reduction of 100,000 in population in the State of North Dakota during the past ten years may be ascribed to this.

CONTROL MEASURES

The expedients used to offset these conditions have been in some cases the construction of additional and deeper wells, the impounding of rivers to increase the underflow and the damming of rivers to preserve as much as possible of the spring runoffs. Decrease in water supplies has forced several of the communities into sewage disposal. While sewage disposal is not advanced as a reason for necessity of increased water supply, yet even with complete treatment of sewage, a normal dilution of the plant effluent is necessary. This dilution does not exist at times in many streams. There appears to be two possible solutions for the supplemental supply for the Red River Valley north of Breckenridge, Minnesota, but only one solution for the valley of the James and Sheyenne River and this solution will also correct conditions in the Red River Valley north of Moorhead.

Earlier in this paper, a chain of lakes was mentioned. Reconstructing conditions on these lakes by adequate damming and reforestration will to a certain extent equalize the flow from that watershed. The watershed is small in area and to a great extent fully populated. The difficulties of this proposal are the objections of the farmers to the flooding of hay lands which have accrued through the receding of lake levels; the objections of resort holders and vacationists to the spoiling of beaches and the fact that the problem is interstate. This watershed is in the same local area and subject to the same variations of season, precipitation and evaporation. An association of seven cities which would be benefited was formed in the winter of 1932–33. Four of the lakes were dammed to the limits which were allowed and the water released during the summer of 1933. The results to date have been only that the lakes below were kept to normal level with no appreciable margin for domestic supplies. This project is therefore quite limited.

DIVERSION OF THE MISSOURI RIVER

A true solution appears to be a partial diversion of the Missouri River. The watershed of the Missouri River above the point at which it enters North Dakota has an area of 165,000 square miles. The flow into North Dakota will average 365,000 acre-feet. The project has been studied since 1921 with the result that the benefits appear to be 100 percent and the evils none. An outline of the proposed project would be somewhat as follows: A dam would be constructed on the Missouri River at a point approximately eleven miles south of Garrison, North Dakota. This dam would be 148 feet above the bottom of the river at that point and would have a length of 11,000 feet between the bluffs on either side of the river. These bluffs rise to a height of 240 feet. The dam would be of the earth fill type in as much as foundation conditions are not suitable for a concrete structure. The dam would be 50 feet wide on the top, 240 feet through at the water line with 6 to 1 slopes on the upstream side and 3 to 1 slopes on the downstream side. The top would be 35 feet above expected water level and the face would be protected with a concrete slab. This dam would form a lake 140 miles long with an average width of one and one-half miles. It would impound 10,000,000 acre-feet. Spillways and sluice gates would be constructed for the passing of the normal, predetermined flow, which would be allowed to pass the year round and also flood flows which might exceed the storage capacity. 80,000 horse-power would be available from the normal discharge from storage. The cost of this dam with the control works is estimated at $43,000,000.00. The dam would control 40 percent of the normal runoff past Kansas City and would control 7 percent of the maximum flood flows. A diversion phase of this project would be accomplished through an intake canal which is an existing stream bed eleven miles in length, a tunnel of 19 miles in length and 14 feet in diameter and a discharge canal of 26 miles in length. The estimate cost of the diversion is $14,600,000.00. At this point, it might be well to note that the normal elevation of the Missouri River is at present 1700 U. S. G. S. elevation. The Coteau-Missouri which parallels the Missouri River on its southeasterly bank has an elevation of from 200 to 220 feet. Devils Lake has an elevation of 1300 feet and the Red River Valley at Moorhead, Minnesota, has an elevation of 900 feet.

Upon completion of the dam and diversion facilities, all flow in

the Missouri other than normal flow, would be impounded. Upon filling of the reservoir, diversion to the amount of 1000 cubic feet per second for seven months of the year would be started. This water would pass through the intake canal, then through the concrete lined of tunnel, through the discharge canal into the head waters of the Sheyenne River. A dam constructed in the Sheyenne River would enable the control of flow through canals into the James River or into Devils Lake at will. Storage and flow could be regulated at will and according to the needs of the various watersheds. A dam could be constructed in the Sheyenne River to divert water by gravity into the Mouse River benefiting Minot and the adjacent territory. It is estimated that in two or at most three years, the water in Devils Lake would be diluted to such an extent as to be fit for treatment and consumption. Devils Lake, which at one time was a field of commercial fishing, now is so highly mineralized through evaporation as to be unable to support any form of fish life. From Devils Lake, which would be raised not exceeding 20 feet, the water would pass through a channel and dam into Stump Lake which would thus be recreated. From Stump Lake the water would return to the Sheyenne River by gravity. Minimum flows in the Sheyenne would be stabilized, the water eventually passing five miles west of Fargo, North Dakota. The Sheyenne River at this point is at sufficient elevation that the water would flow by gravity into the Red River above Fargo and Moorhead through practically a natural channel. Only sufficient water would be diverted into the Devils Lake and Sheyenne River portions of the project to restore the lake and stabilize flows in the rivers. The balance of the water would go into the James River down through South Dakota and return to the Missouri River at Yankton. It is possible to divert water from the James River into the Big Stone and Traverse Lakes, thus supplementing flows in the Bois de Sioux and Minnesota rivers with benefit to communities and inhabitants in their valleys. Sale of power from the hydro-electric plant in connection with the dam, it is estimated, will care for operation and maintenance of the project. The project would be operated in such a manner that storage reservoirs will be drawn to their lowest points for normal flows and diversion purposes each spring and would be refilled with the flood flows in the Missouri above the dam.

The advantages of this project are power, control of flows on the Mississippi, stabilization of flows in the Missouri and Mississippi

for navigation, prevention of the erosion, silting, costly levees and dykes in the interest of channelization, tremendous irrigation possibilities, should the need arise for the future, creation of fish, game and recreation facilities, the providing of domestic and industrial water for the area benefited which consists of 13,000,000 acres and the prevention of a general exodus from the farm and community due to lack of water supplies in this area. The Missouri River water is an ideal water for any purpose. The time required to complete the project is estimated at from two to four years. Labor for upwards of 20,000 men connected directly and indirectly with the project would be provided. This project is not a proposal for new development, but is one to protect existing development. It meets all of the N.R.A. requirements.

As to disadvantages, none of material importance can be found.

The author is indebted to Burns-McDonnell Co. of Kansas City, E. F. Chandler of Grand Forks, N. D., and F. L. Anders of Fargo, N. D., for much of the data given.

(Presented before the Minnesota Section meeting, October 21, 1933.)

WATER WORKS CONSTRUCTION WITH RELIEF LABOR

By Charles Brossman

(*Consulting Engineer, Indianapolis, Ind.*)

I have been asked to speak on water works construction with relief labor, and this cannot be done without referring to the N.R.A. or Public Works.

The $3,300,000,000 appropriated by the Federal Government has been mostly apportioned, but as yet the results in actual construction are not evident. The Gods grind slowly, and they have nothing on Government procedure and caution. However, the delay is probably the result of the gigantic size and intricacy of the mechanism and no doubt it is desirous that the loans granted be conservative and safe.

The Public Works Administration Board in Indiana has had filed with it about 105 to 110 projects of various kinds, 90 of which are now in Washington awaiting approval. Of this number 27 have been approved and include 13 highway grants and 14 municipal projects.

These 27 total about $2,200,000 for the 14 municipal projects, and $850,000 for the 13 highway projects. The total amount involved is about $26,000,000, for all of the projects now in Washington, and this includes a $10,000,000 application on the Indianapolis gas project. The P.W.A. Board of Indiana is one of five or six states that have the reputation of sending applications to the Government in proper form. This state has three engineer examiners, and some other states have from five to six examiners.

What portion of this money will go into labor on these projects? I am going to cite four projects built in 1930 to 1933, totaling in round numbers $250,000 in cost. A complete analysis of costs shows 41 percent spent for labor, and 59 percent for materials. The maximum for labor in one of these projects was 53 percent. One job was analyzed still further and it was found that 80 percent of all the money spent was kept right in the State of Indiana.

Let us take projects, however, that would come under the N.R.A. ruling. I have analyzed and prepared applications for seventeen different projects which have been either sent to Washington or

are in the process of going through. Sixty-four percent of these are water works, the balance sewers and sewage disposal. The total amount involved, in round numbers is $1,400,000.00.

Deducting for overhead, we have	1,283,000.00
Labor	517,428.00
Material	765,567.00

Again we get close to 40 percent of the cost for labor. From this you will note that 40 percent is a fair average figure to use on labor cost as compared to the total cost of a project.

Again with a total cost of $1,400,000, the cities involved will actually pay out about $1,000,000 and receive a Government grant of $400,000 on these projects.

Let us see what this means to the taxpayer. A further scrutiny of these seventeen projects reveals that the weighted average years to pay is 22 years, these projects running from 10 to 30 years.

It will cost the citizens of these cities and towns from $4.80 per capita in the case of extensions or improvements up to $40 per capita total cost when entire new plants are being installed, or an average of $15.20 per capita. In every case, these projects are self-liquidating, and are useful utilities, such as water, lighting, sewage disposal or sewers.

Allowing for the grant, the actual total cost to the taxpayer for seventeen projects, over the period that he must pay is 76 cents per year, or a fraction over 6 cents per month.

Some of these improvements reach as low a cost as 21 cents per year per capita over the period in which they must pay out.

The highest figure is $2.35 per capita per year over ten years, and in this case of a light and water plant improvement the same rates now in effect will pay off the investment.

In practically all of the water works there is no increase in rates, and in some, the improvements, such as at the Bedford, Indiana, water works, the savings in operating costs will practically amortize the investment.

In some of the cases considered the amount paid for poor relief ranges from 80 cents to $5 per capita of population, per year for such poor relief in 1932.

In most cases compared, the poor relief is double or more than double the amount per capita the citizens will pay for these improvements.

Of the seventeen projects for which I have definite figures, these

embrace a population of 68,000 people; in each of these places from 30 to 100 men will be employed from 3 to 10 months, or on the total figure a thousand men for an average of 5½ months, who will receive wages totaling $517,000.

CIVIL WORKS PROGRAM

Some of the things that can be done with relief labor in the water works plant or other utilities such as light plants, sewer systems, etc. are enumerated below.

Repair the walls, roof and physical structures around the plant, repoint brick walls, mend concrete walls, roofs, etc., waterproofing of structures, paint your pump and engine room and boiler room walls. If you have pipe, lay extensions and connect dead ends, pull and clean out wells, if necessary.

On the light plant make repairs the same as those above; if you have wire or cable on hand, string it where necessary, trim the trees where they interfere with wires; creosote the poles and reinforce old poles and replace guy wires.

On the sanitation end, repair sewers and extend small sewers and laterals and make repairs to disposal plant, and get them in good shape.

There are a multitude of such things that can be done with little expenditure. I am not speaking of large projects and extensions, but take care of the minor things under the Civil Works program where you only have to make small purchases and you will find a great deal to do that is really necessary.

Perhaps the World War did not cost eight times as much as the $3,300,000,000 Public Works program, but the latter at least will leave something tangible,—some permanent works of usefulness instead of demolished structures.

This Public Works program is only the band ahead of the parade. It cannot give work to all, nor can it give money enough for the purchasing ability to keep all the various industries moving in a nation that has enjoyed a national income of several hundred billions of dollars.

Public Works can only be the forerunner. It is the means to start the great flywheel moving and jolt this country from its coma of inertia to a moving state of industry and employment. It should prime and move the industrial and financial machinery in this great country of ours.

(Presented before the Indiana Section meeting, December 6, 1933.)

BETTER FIRE PROTECTION THROUGH A MORE EFFICIENT UTILIZATION OF THE PUBLIC WATER SUPPLY

By Clarence Goldsmith

(*Assistant Chief Engineer, The National Board of Fire Underwriters, Chicago, Ill.*)

If public water supplies were not in existence and had not, most of them, grown up piecemeal, it would not be a difficult matter to design and build systems which would carry out with good efficiency their major function of fire protection, as determined by demand in all but the larger cities. Such an opportunity to start from the beginning is very rare, and the usual problem is to take the existing facilities of a system and develop them by carefully studied and reasonable changes into the most efficient means of protection that they can be made to afford. Improvements in efficiency can be made both in the distribution system and in the supply works, and in each of these the improvements may be divided into those which will cost little or nothing to carry out and those which will be more or less expensive. Without doubt, the first group will prove by far the most popular today.

RECORDS

Complete and up-to-date records are conceded to be essential to an efficient water plant. Records need not be masterpieces of draftsmanship, but should be clear and legible, and additions or corrections should never be thrown in freehand or carelessly. The placing of general plan records on tracing cloth, besides giving the chance of filing in duplicate, allows blueprints to be used temporarily for corrections and for many other valuable uses. The furnishing of blueprints of the general system plan to each fire station, particularly if the blueprints emphasize the larger mains by thick lines and are very clear as to all connections between large and small mains, and the careful studying of these blueprints by officers and men will result in more effective utilization of existing facilities, avoiding hydrants on dead ends and 4-inch mains and, in general, effecting the location of apparatus in a direction toward the feeder mains and

source of supply. If, in addition, such maps are supplemented by hydrant pressures plotted at each hydrant and preferably by fire-flow tests of groups at frequent intervals, the fire department can determine definitely whether they may depend upon direct hydrant streams without delaying to connect their pumping engines and for how large a use of water the hydrant streams may be depended upon. Where pressures are lower, such a study is still of value in determining whether the more easily handled soft suctions may be used without danger of collapse. With the weaknesses in pressure and the volume well in mind, the fire department can safely resort to the above speedier methods of operation, except in the known weak localities.

NOTIFICATION OF FIRES

Definite arrangements for notification of some responsible waterworks employe of all serious fires and response to these with suitable tools and field records are a considerable aid in efficient utilization, or better, in conservation of available supply. In extensive fires, falling walls or floors often break off sprinkler or large service connections and these, if left running, will bleed the system of large quantities of water needed for extinguishment of the fire. Occasionally shifts in wind will necessitate abandoning hose lines and open hydrants which can only be shut off by the system gate valves. In addition, the waterworks' representative, with his more intimate knolwedge of the system, is in a position to advise on the proper location of additional apparatus, to relocate any which may be misplaced and draining supply from another, and to correct any defects in the condition or use of hydrants, such as one operating stiffly being used only partially open.

REGULAR INSPECTION OF HYDRANTS AND VALVES

Efficient utilization of an existing system demands that it be in first-class condition when called upon to furnish fire protection. This requires regular and thorough inspections of all hydrants and gate valves and prompt repair of all defects found. There is little that will arouse greater criticism of a water department than to have a hydrant frozen or inoperative when its use is attempted at a fire. Complete inspections twice a year, with especial attention to drainage, and a complete record and subsequent inspection of all used during cold weather should insure that each hydrant will be ready to serve its purpose when needed. Special conditions of ground

water may require the pumping out of hydrants or even the placing of an anti-freeze solution in the barrel. Inspections of gate valves are equally important to insure full efficiency. When a shut-off must be made because of a broken main or a hydrant broken off by the falling wall of a burning building, it does not appear very efficient to have the "geyser" spouting for half an hour while the crew struggles with a wedged box cover, spoons out a box full of dirt, digs up a box which has been pushed away from the operating nut, or finds the stem stuck or frozen so tight that they break the stem in the effort to start it, and have to move back two or three blocks and repeat the whole process. As long as men are human a valve will occasionally be left closed, and closed valves can only be discovered by accident or by inspections. All valves should be inspected; a closed 6-inch valve or a defective one which allows a 6-inch stream to waste is just as important, if located close to the seat of a fire, as any of the larger ones. For the smaller valves, inspection once a year should be sufficient, but geared valves, unless the gears are oil-encased, will require more frequent attention. The very distinct marking, both on the box wall or cover and on the records, of any valves which operate in a direction opposite to the majority is important to avoid confusion, and such valves should be changed to conform to standard whenever any repairs are required.

There are some improvements in efficiency which can be made at little or no cost. Usually someone in the organization can do the work of making adequate records when other work is slack. Often a "fire fan" on the force will welcome the opportunity to attend all large fires. Inspections can be made at times when normal work is light. Repair costs may be rather heavy at first if inspections have been neglected, but should not continue to prove too expensive. Other improvements in the efficiency of the distribution system involve more expense, but some may be included in the financial set up as the maintenance costs, which they really are. Nearly every system has a number of hydrants which cannot be classed as anything but inefficient and, unfortunately, these are often located in the more closely-built or even in the high-value sections. Defects may be in the size of the branch or in the hydrant itself, introducing unnecessary friction loss when drawing water; there may be no suitable large connection for the use of fire department pumping engines, or the type of hydrant may be obsolete, so that repair parts are not available and costly local manufacture is necessary. Although the

hydrant is the final link in the chain between a supply of water and fire extinguishment, it is as important as any other, and such inefficiencies should be overcome as rapidly as possible. A program should be outlined for the replacement of a definite number of inefficient hydrants each year, beginning in the central business district and in other important sections and gradually extending to all.

Spacing of gate valves is another feature having considerable bearing on this subject. It is not efficiency in domestic service to shut down four or five, or even fifteen or twenty, blocks of the system to make a repair in one of them, nor is it efficiency in fire protection if a broken hydrant must be allowed to run because shutting it off would also affect most of the other hydrants being used for the fire. It is usually where such long shut-off sections are found that valves are not provided in hydrant branches either. Again, a regular program for improvement is the logical thing. This should be planned by tracing out with colored pencils on a blueprint of the system the lengths which exceed the reasonable maximum it is felt can be left and by noting the longest ones or any which involve shutting down feeder mains for first attention.

BAD DISTRIBUTION MAIN LAYOUT

The mains of the distribution system may not be laid out to deliver fire protection to all portions efficiently. This may be because the smaller mains are not connected to the larger, and this easily can be remedied by installing side or "run-around" connections. In general, the side connection is a very good method of connecting mains which cross, requiring fewer gate valves and allowing uninterrupted supply through either main in case of a shutdown in the other. The running of feeders "express" through a district without any connections is rarely justified except for separate services or in very large cities; for if there be danger of pressures being reduced under usual conditions in the feeder by the local connections, then that size of feeder is needed for that special district and a larger size should be provided to supply any districts beyond adequately. Considering added efficiency in mains also involves the closing of any gaps in feeders, the extending of subfeeders from the large mains into districts not properly supplied, the cross-connecting of long, unsupported gridiron lengths which may be parallel to and only a few blocks from a large main, and, of course, the elimination of all dead ends possible. Rearrangement of service limits is often possible to eliminate dead ends

and bring the greatest number of hydrants possible into whichever service is stronger in available volume, not necessarily in pressure.

Next comes the case, and unfortunately it is a fairly frequent one, where a system fails to deliver the quantities which all our theory and mathematics indicate that it should. This happens as frequently in the strong systems which have had careful attention to other features of distribution as in those which have not had such attention. The first suspicion is that of closed gate valves, but with that eliminated through careful investigation and inspection the discrepancy can be assigned to only two causes: some unknown obstruction, or the retarding influence of incrustation and tuberculation built up by water of unfavorable chemical content. There are then two alternatives for correcting this inefficiency: either by resigning yourself to the fact that the existing main must be considered as reduced one or more sizes in capacity and laying additional cross or even parallel mains, or else by cleaning the mains. This second alternative generally has proved the most economical. When a large portion of the system is more or less affected by incrustation, once again a planned annual program is the logical procedure for improvement.

SUPPLY WORKS

Turning to the supply works of a system we find much less frequent opportunity for efficient utilization with little or no cost, but there are a few. Meetings of operating employes may be held at which the proper operation of the system may be discussed, and the outline of the most efficient method of operation in any break of equipment or piping or in any other emergency may be brought clearly to the mind of each employe. Many plants have by-pass lines or duplicate supply arrangements; for example, in boiler-feed piping, but often these are rather complicated, necessarily so when added on to existing piping layouts which may have been added to piecemeal and which originally were not laid out to the best advantage. A clear outline of just what must be done in each emergency will often be a great aid in speedily and efficiently overcoming it.

PUMPS

A condition not occurring very often, but of such importance when it does occur as to be well worthy of mention, is the efficient

utilization of pumps. Plants may have centrifugal pumping units which have been used working in series against fire pressure. As modern practice progresses and fire pressure is discontinued, with the introduction of elevated storage or otherwise, these pumps become useless for their original purpose, yet the available capacity from them remains half of what might be obtained. Let us take for an example a set of pumps designed to deliver capacity at 100 pounds fire pressure or 50 pounds for each, and each not able to deliver at over 55 pounds. Elevated storage requiring a normal pressure of 60 pounds is installed and the pumps must still operate in series and at their original capacity, but very inefficiently. By the simple and comparatively inexpensive expedient of changing pump impellers, these pumps can be operated in parallel and make available double the capacity for reserve; if motor sizes were originally skimped they may not be sufficient, but usually the margin of safety provided will be sufficient to provide for the added load.

Efficient utilization of reserve equipment and particularly steam equipment cannot be expected unless it is maintained in good condition. To insure this, it should be turned over monthly and operated to capacity at least every six months. This has the added value of refreshing the details of procedure for the operating crew, which, without this, may easily lose part of the routine and smoothness of detail that are the essentials of good operation.

Inprovements in the reliability and consequently the efficiency of the supply works may be numerous and varied, and the cost may also vary considerably. The use of the entire remainder of the system may be hampered by lack of sufficient reserve in pumping capacity, either high- or low-lift, or in generators or boilers. Cases have occurred where the whole adequacy and efficiency of the plant were limited by the size of the stack provided. All piping upon which supply is dependent should be studied, and the effect considered, first, of a break and then of the repair of any valve. This will include both high- and low-lift suction and discharge piping, steam and boiler-feed lines, fuel lines to boilers, of other than solid fuel is used, air lines with air-lift equipment, and, falling into the same classification, although not strictly piping, the electric supply lines. In some cases, simple changes or the addition of a few valves will improve reliability materially, and in others a good deal of study and ingenuity are needed to accomplish this.

SUMMARY

Let us summarize these rather diverse suggestions for improved efficiency in the utilization of a water supply: First, and most easily done, is to make the very best possible use of the facilities already provided. This necessitates the close cooperation of the fire department, but must be aided by the provision of proper facilities for studying the system and augmented by water department representation and advice at serious fires. Next is the insuring of efficiency in the use of the system by keeping all portions in the best possible condition. Last, are the changes and additions which must be made in a system to give and to insure at all times the full utilization of all the costly fire protection investment already made. With the first two of these carried out and with the many possibilities of the third carefully studied, plans made and a program outlined to agree with financial limitations, a superintendent can feel that he has done his full duty toward the efficient utilization of his water supply for fire protection.

(Presented before the Indiana Section meeting, December 6, 1933.)

COLLECTION OF WATER BILLS DURING A DEPRESSION

By H. A. Dill

(*Superintendent of Water, Richmond, Ind.*)

The public generally has the impression that a utility is a heartless corporation and is always ready to exact its "pound of flesh." Is that correct?

Of course, utilities are generally empowered by law or by approval of State Commissions to adopt fair rules relative to collection of bills. All operating companies know from experience that extenuating circumstances may necessitate some modifications of the rigidity of such rules, without intentionally violating their fundamental intent.

During the past two or three years, a most unusual condition in collections has arisen. Unemployment has been so widespread that a large percentage of customers have not been able to meet their obligations of various kinds, including water bills. These families may already be receiving aid for food, fuel, clothing and housing. In many cases, this number may be as high as 25 percent of the population. Pure water has become a requirement for the health of the families themselves, as well as for the community.

Shall a water plant, private or municipal, recognize, therefore, the emergency conditions, and be permitted and expected to modify the rules concerning collections from indigent consumers? Public opinion would demand it, for reasons of humanity as well as for protection of health. There are few wells in a city that would pass the test for purity. Therefore, water must be obtained by indigent families from sources that are potable and sanitary, or be given the use of city water under varying conditions existing in different communities.

No fixed rule can or should be made for all plants. The matter should be left to the judgment of the officials in charge of each unit. Each case involved should be handled on its merits. Applicants may be referred to local relief organizations which may have funds to pay part or all of bills due, the Company may accept partial payments, or may arrange to furnish company work projects to

cover bills due and delinquent. Investigations should be made to determine the ability of consumers to pay. Landlords may voluntarily pay the bills of tenants, but there is no law in this State requiring this, unless they have signed applications for service or have guaranteed the accounts of the tenants.

EXPERIENCE IN INDIANA

How have water companies in this State been handling this problem of indigent bills? A questionnaire was mailed to 26 towns and cities to obtain their policies, and the replies are interesting and surprising. An analysis only will be given, as the information given may be confidential.

1. Do you enforce your regular rules on delinquent bills?

21-replies—19—No
2—Yes

2. If not, what is your present policy?

18-replies—17—Accept partial payments, dependent upon credit based on previous payments.
Send out collectors to obtain what is possible.
1—Allow accumulation of delinquency to a determined amount or time. Each case considered separately.

3. Do you charge for turning on in all cases of delinquent bills?

21-replies—17—No.
1—Yes
1—free turn on a year
2—Certain cases, No.

4. What percentage of total revenue was delinquent and unpaid November 1, 1933?

15-replies— 2—$2\frac{1}{2}$
1—5
3—8
3—12
1—15
2—20
1—$37\frac{1}{2}$
1—50
1—$570.00 on 164 accounts.

5. Do any relief organizations pay all or part of bills of consumers on poor relief?

20-replies—17—No
3—Yes

6. Does your Township Trustee pay any water bills under a Law passed at the 1933 Session of the Legislature applying to Poor Relief?

21-replies—17—No
4—Yes

7. Do any of your consumers "work out" their bills on work for the Company?
20-replies— 8—No
11—Yes
1—Very few

8. Do landlords pay or guarantee water bills for tenants?
20-replies—11—No
4—Yes
1—Where two or more in building
2—Pay, but not guarantee
2—In some cases

9. Are deposits required and how much—(domestic only)?
19-replies— 5—No
1—$1.00 to $5.00
2—$5.00 tenants only
1—$2.50
5—$3.00
5—$5.00

From this analysis, it will be seen that water companies, municipal and private, are doing their part in contributing to the welfare of the communities in this State, and it would seem proper that the public should be advised of this fact. Most of the delinquent bills will never be paid for various reasons, and will be charged to profit and loss. It is and will be a source of satisfaction to water company officials that they have not allowed their desire for a good financial showing to supersede the humanitarian element in this period of depression.

(Presented before the Indiana Section meeting, December 6, 1933.)

THE LEGAL ASPECT OF WATER WORKS ADMINISTRATION IN CANADA

By R. C. Harris

(Commissioner of Works, Toronto, Canada)

In dealing with "The legal Aspect of Water Works Administration," I shall necessarily have to confine my observations largely to the law governing the operation of water works activities in urban centers, but hope to discuss some phases of the problem which may be of assistance to those charged with such duties in suburban or rural districts.

The construction and operation of water works facilities is governed by legislative enactment and any powers exercised by the municipality by by-law or regulation are derived from the Provincial Legislature by way of general law, or special act relating to a particular municipality.

In considering the theme under review, it is well to establish a proper background, in order that the phases of the problem discussed may be adequately related.

THE HISTORY OF THE TORONTO WATER WORKS

In the year 1841, a private company was incorporated under the name of "City of Toronto Gas, Light and Power Company" for the purpose of supplying water to the inhabitants of the City. The name was changed to "City of Toronto Water Company" in 1853. In that year another company was incorporated with the title "Metropolitan Gas and Water Company." The latter subsequently purchased the City of Toronto Water Company of which Albert Furniss of Montreal was proprietor and sole shareholder. Furniss was given a mortgage on the assets of both companies and eventually foreclosed. By Private Act passed by the Provincial Legislature in 1872, all the powers and assets of both companies were vested in Furniss and he was empowered to conduct the business under the name of "City of Toronto Water Company." In the same year, by 35 Vic. Cap. 79, "The Toronto Water Works Act," a private bill, was enacted, authorizing the City to construct, acquire and operate a

water works system. Furniss died. His heirs sold the business and equipment of the water company to the City, which sale was confirmed by statute in 1874, and the municipality conducted the business of the project through a Commission.

The preamble to the Toronto Water Works Act is of interest, in that it sheds some light on the reason for the City embarking upon the venture of supplying water. The preamble to the aforesaid Act reads as follows:

"Whereas grave and frequent complaints have been made from time to time, by the citizens and Corporation of Toronto, against the quality and supply of water furnished by the Toronto Water Works Company, hitherto existing and supplying water to the city, and grievous and serious injury to property and to the city generally has resulted from an undue and insufficient service thereof, and whereas numerous amendments have been made to the charter of the said Toronto Water Works, with the view to making the same more useful and effective, for the purposes intended, and to enable the Corporation of the City of Toronto to satisfy the citizens as to the service of water procurable from said company, and whereas after much treaty and negotiation between the said City of Toronto and the said Water Works Company, it has been found impossible to effect any satisfactory arrangement with said Water Works Company on behalf of the said corporation and citizens of Toronto, and whereas the Council of the Corporation of the City of Toronto, have by petition declared that it is deemed necessary and advisable that the said Corporation of Toronto should have the power to purchase, construct, have and manage, as to them may seem meet, certain water works on behalf of the City of Toronto, and it is expedient to grant the prayer of said petition:" etc., etc.

The Toronto Water Works Act of 1872 has been amended on various occasions. Originally it provided for the administration of the water works by five commissioners. In 1878, legislation was granted which transferred the powers and duties of the commissioners to the City, and the commission was abolished.

The Act confers all powers necessary to operate the water works and purchase the works of any other company, and to improve, secure, maintain and enlarge any of the said works from time to time; the agents, servants or workmen at such times as may be seen fit, may enter the lands of any person or persons, bodies, politic or corporate in the City of Toronto or within thirty miles of the City; to divert springs and streams; to acquire by purchase, lands, easements or water rights; to erect reservoirs, water works, plant, etc., on such lands and convey water thereto or therefrom in, upon or through lands lying intermediate between such reservoirs, water works, plant, etc., and springs, streams, rivers or lakes from which same is

procured and the City of Toronto; and for such purpose to cut up and dig said lands and lay down pipes, through same and in, upon, over or under the highways, railways and roads of the Townships of Etobicoke, York and Scarborough and the Village of Yorkville and the streets, etc., of Toronto and the lands of any person or persons whatsoever; to alter, repair and replace pipes, plant, etc., subject to doing as little damage as possible and making reasonable and adequate satisfaction to the proprietors of the lands affected.

Chief Justice Strong, in Attorney General of Canada v. Toronto, 1892, 23 Supreme Court Reports pp. 514, in construing the Toronto Act said: "The water works were not constructed for the benefit of the ratepayers alone, but for the use and benefit of the City generally whether taxpayers or not." He goes on to refer to a provision of The Municipal Act which no longer exists but his remarks are still applicable. "That provision makes it a duty obligatory on the City to furnish water to all who may apply for it, thus treating the corporation not as a mere commercial vendor of a commodity, but as a public body entrusted with the management of the water works for the benefit of the whole of the inhabitants and compelling them, as such, to supply this element, necessary not merely for the private purposes and uses of individuals, but indispensable for the preservation of the public health and the general salubrity of the City."

GENERAL AUTHORITY

The Ontario Provincial Legislation from which municipalities derive authority to install, operate and maintain water works are: The Municipal Act; The Local Improvement Act; The Public Utilities Act; and The Provincial Health Act, together with such private bills as have been passed conferring specific power upon certain municipalities.

Under existing legislation in Ontario, the duty to supply wholesome water is imposed upon the owners of occupied premises and there is no statutory duty imposed upon municipalities requiring them to supply water, or provide wells, reservoirs or water works.

The general Acts bearing on the question of water and water supply and the establishment of water works are permissive only. The mere vesting of the power still leaves with the corporation, discretion to act or otherwise as the councillors may dictate. In the event of the corporation as a municipal entity declining to provide a water service, the remedy lies with the electors in that they may displace

their elected representatives and elect others prepared to do their will.

However, once the responsibility of supplying water is assumed, the provisions of the Public Health Act apply. The Provincial Department of Health under this Act, has general supervision over all sources of supply; can restrain pollution of the source by injunction, and prohibit the deposit of filth in any lakes, streams, etc., which may affect deleteriously the quality of the supply. The aforesaid Department also controls the disposal of sewage, and the repair and operation of the water works is subject to provincial direction and regulation.

Appended to the Public Health Act is a statutory by-law operative in all municipalities, which has the same force and effect as if it had been enacted by the municipality. This by-law states that it shall be the duty of the owner of every house within this municipality to provide for the occupants of same a sufficient supply of wholesome drinking water, and if the occupant of the house is not satisfied with the wholesomeness or sufficiency of such supply, he may apply to the Local Board of Health to determine as to same, and if the supply is sufficient and wholesome, the expense incidental to such determination shall be paid by such occupant, and if not, by the owner, and in every case, such expense shall be recoverable in the same manner as municipal taxes.

The Act also provides that municipalities shall, as required, make report to the Provincial Board of Health. A penalty is provided for default.

COURT DECISIONS ON CONTAMINATED WATER

The Courts have held that a contaminated water supply is a nuisance. The Supreme Court of Canada in Dominion Canners v. Costanza, 1923, Supreme Court Reports pp. 46, declared "Any well, spring, or other water supply injurious or dangerous to health is a nuisance."

In Campbell V. Kingsville, 37 Ontario Weekly Notes pp. 51, recite a judicial finding in the following words, viz.:

> "A municipality which knowingly maintains a dangerous water supply is answerable for damages to all persons who suffer ill health or contract disease by drinking water from such source."

From the foregoing, it is apparent that once the municipality has undertaken supply, it must be potable in quality and so maintained.

OTHER LEGAL REQUIREMENTS

It has been held that after having established a water service for fire protection, the municipality is obligated to maintain same, but in the case of Alexander v. City of London, 15 Ontario Weekly Notes at pp. 20, it is indicated that aside from negligence, there is no duty to maintain pressure for fire protection upon which an action for damage or loss by fire can be founded.

The Courts have adjudged that the duty of the City of Toronto to supply water is statutory and not contractual. The Scottish Ontario Manitoba Land Company instituted an action against the corporation, for alleged damage to an hydraulic elevator by reason of the presence of sand in the water supply. The plaintiff plead that the City was liable for damages for breach of contract to supply pure water. Justice Osler in appeal held that, if the supply of water under the statute is strictly a municipal function or duty and not a mere power conferred upon the City as a corporation, a contract could not be spelt out of the request made to the City by the plaintiff for water supply and the City's compliance therewith.

In this case, the potability of the supply was not in question, but rather the presence of sand which, it was alleged, damaged the operating mechanism of an hydraulic elevator.

The Public Utilities Act applies to all municipal water works, except in cases where it is inconsistent with a private Act governing individual municipalities.

Apart from the provisions of the Public Utilities Act and any private Act which authorizes the laying of mains, etc., under highways, there is no special statutory provision with respect to the duties and liabilities of municipalities in respect to water mains, etc., situated under highways. The sections of the Municipal Act dealing generally with the non-repair of highways and the decisions thereunder, apply in actions for damages alleged to have been sustained on account of highway conditions due to the placing of watermains, etc., and state of non-repair resulting therefrom.

INTERESTING COURT DECISIONS

There have been some interesting Court decisions which are especially informative. In Odell v. City of London reported in 17 Ontario Weekly Notes, pp. 284, the plaintiff alleged that a water main burst due to frost, and flooded his premises with resultant damage. The Court held that the City of London was negligent:

(a) In the maintenance of a hydrant in a manner which prevented the proper functioning of the frost jacket.

(b) By reason of failure to keep hydrant free from snow and ice whereby it was prevented from operating properly.

(c) By unreasonable delay in shutting off water after notice of break.

(d) By reason of failure to maintain a proper system of men and appliances to attend promptly to breaks during cold weather, and the provision of proper plans to enable repair men to locate valves without delay.

Some years since, proceedings were instituted against the City of Toronto by a property owner adjacent to the High Level Pumping Station, who claimed that vibration from the operation of pumping machinery had been harmful to the health of his wife. It was averred that vibration was particularly noticeable when the wife of the plaintiff was abed, and that her rest was disturbed thereby. Frequent inspection and test did not disclose to myself and others any trace of the vibration complained of. The Court, however, held that the plaintiff was entitled to damages.

In the case of Attorney General of Canada v. Toronto, Justice Patterson held that, although there was a duty to supply water, there was no duty to supply free of charge, or free from restrictions as to quantity to be used, or the mode in which, or the purposes for which it may be used. In this case, the City sought by by-law to except Dominion Government Buildings from the benefit of a discount on water accounts, allowed to water consumers generally. The by-law was held to be illegal in that it was discriminatory.

The Toronto Act and the Public Utilities Act provide that water rates shall be a lien on the land and payment may be enforced by discontinuance of service or distress. It also provides penalties for injuring plant, unlawfully laying pipes and draining off water from source of supply, sale, distribution or wastage of water, and damaging or altering meters. It further asserts that the Corporation shall not be liable for damages caused by the breaking of any service pipe or attachment, or for shutting off water to repair mains or tap pipes.

EXTENSION OF MAINS

The Ontario Local Improvement Act provides that the construction, enlargement or extension of a water main, including a main on each side or one side only of a street, may be performed as a Local

Improvement subject to the provisions of the Act. Until 1921 water mains in Toronto were not laid under Local Improvement process. In that year, the following question was submitted to the ratepayers at the annual municipal election, viz.:

"Are you in favour of laying water mains for the supply of water for domestic purposes under the provisions of the Local Improvement Act?"

The vote registered was 39,136 in favour and 6,809 against, whereupon the Municipality secured from the legislature, a Private Act, authorizing the laying of new water mains under the provisions of the Local Improvement Act, with the stipulation that the whole of the cost thereof, may be levied by a frontage tax on the properties fronting or abutting on the work, and otherwise, pursuant to the provisions of the Local Improvement Act. You will observe that this enactment permits the whole of the cost of the improvement to be assessed upon abutting frontage, while under the terms of the General Act, the City would otherwise be compelled to absorb its statutory proportion of the cost.

Objection has been raised from time to time, that vacant lands on streets upon which a water main is laid by Local Improvement process, should not be charged for a service which would not be required until buildings be erected upon such lands. It is patent, however, that the availability of water imparts a decided increment to the value of land, and to my mind there is not any just reason why property so benefited should not be assessed therefore.

Water mains may be laid under the Local Improvement process by: (a) Petition; (b) Initiative recommendation; (c) Section 8 of the Local Improvement Act known as the "force" section, and (d) Section 9 of the Local Improvement Act where the work is initiated as a sanitary measure by the Local Board of Health.

The procedure governing the process is fully set forth in the Local Improvement Act and it would unnecessarily infringe upon your time, if I attempted to detail the various and somewhat complicated steps, necessary to the consummation of the Local Improvement process.

I desire to emphasize the necessity for observation of meticulous care, in the institution, sequence and consummation of all legal requirements, in the design, construction, and operation of a water works utility, as the Courts have held that the omission of a single purely formal requirement, invalidates the whole procedure.

Concluding, I record my appreciation of the assistance of Mr. R. C. Baird, Assistant City Solicitor for Toronto, without whose good offices the preparation of this paper by one not skilled in law, would have been impossible.

(Presented before the Canadian Section meeting, March 24, 1933.)

WHAT THE MARSHALLTOWN WATER WORKS DID THIS YEAR

By H. V. Pedersen

(*Superintendent, Water Works, Marshalltown, Ia.*)

The fiscal year of the Marshalltown, Iowa, Water Department ends March 31. Last April after the yearly financial report was completed the figures revealed that the income for the year was $20,000 less than the year previous. Furthermore, the figures indicated that the possibility of the income increasing during the next several months to come was rather slim. A comparison of the figures on this report with the figures on similar reports of past years showed that the decrease of $20,000 in income represented the amount spent yearly as capital improvements.

I doubt if the financial status of many other water works was little better off if the truth were admitted. Probaby almost every water superintendent and manager asked themselves the same question I did. Certainly the prospects of constructive improvements was about as dark as during the darkest ages.

COAL LOADING ARRANGEMENTS

But it so happened that about the time when the Water Board, with the aid of the self appointed tax reducing enthusiasts, decided not to spend a nickle, the coal elevator at the pumping station, which elevated the coal into the bins and which because of the topography of the area surrounding the plant was absolutely necessary, broke down. A fine state of affairs this was. Just when it had been decided to save we had to start out spending or pile the coal for the plant out in the driveway. Well if it had to be it had to be, tax committee or no tax committee, money or no money.

I set about obtaining quotations on coal elevators and began planning on a new installation. Then one spring-like morning as I was looking over the old broken-down elevator I got a bright idea. I conceived the idea of building an overhead viaduct for unloading coal instead of buying a new coal elevator. I drew up a set of plans and made an estimate of costs. I then took each member of the water board alone down on the job and told each one individually

that it was just about as cheap to build a concrete bridge as it was to buy an elevator. A viaduct 50 foot long with a roadway 10 feet wide would just span the gap between the top of the coal bin and the hillside to the east. I also pointed out how it would be possible to greatly improve the looks of the steep hillside which had usually grown up in weeds. At a special meeting of the Board I was instructed to go ahead if I was so sure I knew what I was talking about.

I began work on April 18, superintending construction in person, and finished June 15, 1933. As the work progressed the possibility of improving the looks of the grounds became more apparent. With just a little more expense a retaining wall could be built to reduce the steep grade. I took it upon myself to fix up a small section as I wanted it before I called for inspection. During the inspection I was agreeably surprised by being told why I did not fix up the rest of the hill to look like something. We fixed it up. We built a new hard surface road to connect the street to the new coal viaduct. We built retaining walls to reduce the grade of the hillside and sodded the newly acquired lawn.

We spent approximately $2,000, $1,000 on the viaduct and $1,000 on the road and retaining walls. A good coal elevator was priced installed at $1,100. By building a viaduct it has made it possible to save six hours in unloading a car of coal. I estimate that a total saving of $200 a year will be made over the use of the elevator. In five years the added cost of improvement will be paid for by the saving, to say nothing of the many favorable comments received as a result of the improved looks.

NEW PUMP INSTALLATION

We had just about finished the viaduct when the fire underwriters inspector paid us a visit. During this inspection we were told that unless we installed a stand-by pumping unit, which I had been talking about for the past six years, the fire rates might have to be increased. This did not hurt my feelings one least bit, but it did not sound like music to the ears of the self appointed tax reducing committee. But the Board of Water Works Trustees by this time had had a change of heart, and decided it was as good a time as any and started in. We bought and installed a 2 m.g.d. capacity centrifugal, electrical driven pump operating against a head of 225 feet. We installed this pump in the old original pump pit built back in 1890 which had been standing idle for a long time. This improvement, including

the pump, pump discharge line, electrical apparatus and fixing up the old pit, cost approximately $3,500. The unit has been tested out satisfactorily and it will pump water at a cost of 1 cent per 1,000 gallons for the electrical energy. The coal used in the steam plant for the past six years has averaged $\frac{3}{4}$ cent per 1,000 gallons of water pumped. (I still believe in good old steam as the most reliable and most efficient power for pumping water.)

EXTENSIONS OF MAINS AND OTHER REPAIRS

After we had finished with the new pump job we decided to lay several blocks of water main in order to give a certain class of laborers a chance to work out their water bills. We also decided to replace a few old flat head Chapman hydrants that had been giving trouble. In working around some of the old hydrants we found conditions which decided us to inspect all of the old hydrants carefully and to repack them. For the last two months my maintenance crew has been repairing and painting hydrants. We have been painting with aluminum and so far we have been well pleased with the results.

We also painted our 100,000 capacity elevated steel tank. Three young dare devils, local Junior College boys, made me a proposition and I took them up on it. Parts of the tank were in bad shape. We first scraped the surface clean and gave it a coat of red priming paint. Then we covered this with a coat of aluminum. Up to now I have not been entirely sold on aluminum paint and I still have to be convinced. I had, however, been watching several tanks in the vicinity of Marshalltown for three years that had been painted with aluminum. The paint jobs are apparently in such good shape that I was willing to risk it on our tank.

Back in 1900 when the city acquired a certain tract of land for water works purposes they acquired a house. The house is located on top of the hill overlooking the plant and is used by our chief engineer. During this summer my attention was called to the condition of the foundation of this house. When called to the attention of the Board of Trustees they decided it was a matter of economy to repair the old house. We took out the old foundation, dug out a cellar under the entire building and constructed a new foundation. We also installed a furnace and repaired a lot of the plumbing.

During the summer the income of the department increased a little, although the number of unpaid accounts remained about the same. In order to clean up some of these accounts we decided to do a little work on which we had never felt justified spending good hard

water works money. We set a gang of men picking up stones from the Iowa river bed and piling them along a concrete retaining wall which the water was undermining. This work brought forth so many favorable comments that it was decided to clean up the river bed all along the water works property and to riprap the part of the river bank that was not protected by a concrete wall. We have been working from ten to twenty men at a time replacing them as fast as a bill was worked out. We have worked in all 125 men and have cleaned off the books approximately $1,000.

In addition to the work mentioned we have done considerable painting on the various water works buildings. We have also taken the time to go over all of our buildings and filled all the cracks and openings around doors and windows frames with a caulking material. We have used the caulking material made by the Tremco Company and I want to say here that it is a wonderful material for this purpose. I also want to emphasize the advisability of the use of caulking. It keeps out the rain which in turn eliminates the danger of frost action and also preserves the inside walls. More damage is done to a brick building as a result of moisture getting down through the coping than any one thing and our experience is that it pays to keep out this moisture by caulking all the joints and cracks.

Taken all in all I have been just about as busy this year as I have ever been. We have made new improvements representing an expenditure of $8,000, $1,000 of which represents working out unpaid water bills. And the best part of it all, we have received only favorable comments for our efforts to keep the unemployed busy. This winter we are planning on making a river intake hook-up to our plant. By building a good intake and connecting up a pump to our settling tank we can secure an excellent stand-by unit which might come in handy in case anything should go wrong in the well field. Most of the expense of this project will consist of labor and is designed to keep my regular crew busy all winter.

At this writing we have a surplus of $10,000 in the General Fund which can be used for capital expenditures, provided nothing happens which will compel us to use it for maintenance. We find that things looking a little better than they did a year ago. Unemployed laborers are looking hopefully for a job next spring and we are looking forward to a fairly good summer next year.

(Presented before the Missouri Valley Section meeting, October 27, 1933.)

THE WATERWORKS OF WILMETTE, ILLINOIS

By J. W. Nemoyer

(*Assistant Engineer, Pearse, Greeley and Hansen, Chicago, Ill.*)

The Village of Wilmette, Illinois, is a strictly residential community situated on Lake Michigan just north of Chicago. It is served by the Northwestern and North Shore railways and by the Chicago Rapid Transit Company. There are no industries and but few stores, garages, etc. Wilmette has a population of slightly over 15,000 people.

As in the case in other suburban communities along the shore line north of Chicago the demand for water during the summer months is high.

Hitherto Wilmette has purchased filtered water from Evanston, which city lies between Wilmette and Chicago. In November, 1931, the voters of the village approved the issue of Water Revenue Bonds to finance an independent water works for Wilmette.

Bids for the construction of the works were received by the Village on April 5, 1932. It was planned to receive bids on the bonds at that time. There being no bids on the bonds the bids for construction were rejected. There followed a period of negotiation with possible private purchasers of the bonds and with the Reconstruction Finance Corporation. Following agreement by the R. F. C. to buy the bonds, bids were taken for the construction of the works on November 15, 1933. The November bids totaled \$394,365.00 or approximately \$60,000.00 less than the April bids.

After bids were received, in November, suit was brought to prevent the Village from constructing the plant on the basis that it would constitute a nuisance. The bill of complaint was filed with the R. F. C. by the Village. The R. F. C. investigated the matter thoroughly and in February granted the Village moneys with which to proceed with the work. The work was actively started in March and construction is proceeding as well as might be expected in consideration of the weather.

The site of the Wilmette Waterworks about centers on the public bathing beach. It is located in the midst of fine residential develop-

ments. It will stand just east, toward the Lake from the Shawnee Country Club.

In view of the environs of the waterworks, every effort has been made to avoid detracting from the surrounding developments. The roof of the works will be mantained about even with the top of the bluff into which the plant is partly built. This will make toward an unobstructed view of the beach and lake which the club now enjoys. The building has been so designed as to harmonize with its surroundings. The roof of the building will form a promenade deck with an area about 100 by 275 which will be tiled. The roof construction includes an ornamental fountain. The building includes sufficient space to provide for a bath house in the future which will permit the removal of a metal bath house now existing. This feature alone will make for considerable improvement in the appearance of the neighborhood.

During recent years the average annual consumptions of water in Wilmette has been about 500 million gallons or 90 g.p.c.d. Summer peak demands have been as high as 400 percent of the average. In determining the capacities of the various elements of the water works consideration was given to the fact that water consumption in the North Shore suburbs has increased materially as the supply of good water has increased. An actual water shortage in Wilmette during the summer months has limited the potential water consumption to a marked degree. Average annual water consumptions per day in north shore communities similar to Wilmette are as follows:

PLACE	G.P.C.
Kenilworth	146
Winnetka	161
Glencoe	237
Highland Park	172
Lake Forest	240

Peak summer demands in these communities sometimes reaches 500 percent of the annual average.

The Waterworks for Wilmette comprises the following:

An intake is being constructed to extend 2,700 feet into the lake. The intake pipe will be electrically welded steel pipe 33-inches in diameter and will have a wall thickness of $\frac{5}{8}$-inches. The pipe will be dipcoated with a bitumastic solution. The pipe will be shipped

and laid in sections 120 feet long. Joints in the pipe line will be made under water using Victaulic couplings. The pipe will be laid below the bottom of the lake and will terminate in water 30-feet deep. Two intake drums equiped with wood screens will be constructed at the outer end of the intake line.

A four compartment intake well about 30-feet deep will be constructed adjacent to the pumping station. A 20-foot drawdown below lake level is available. Provision has been made so that sludge from the sedimentation basins and/or wash water from the filters may be returned to the intake well during periods of low turbidity in the lake water.

The low lift and high lift pumping stations, the filter plant and the space provided for future bath house are all included under one roof.

The low lift pumps include two units rated at 4.0 each and one unit rated at 6.0 m.g.d.

The high lift pumps include five units, graduated in size, having a total rated capacity of 12.0 m.g.d. The pumps were specified as to capacity by a head-discharge requirement. That is to say the total discharge of the combined units was specified for heads ranging from 147 to 197 feet. The motor capacities were specified definitely and the pump manufacturers were allowed to make their best proposals for the various pumps to most nearly meet the nominal capacities of the motors. This resulted very satisfactorily in good, efficient pumping units being offered. Power from two sources will be brought into the plant.

Wash water for the filters will be supplied by a pump having a variable capacity of from 15 to 20.0 m.g.d.

Two reaction basins, each having two compartments, will have a total capacity of 160,000 gallons, giving a retention period at 6.0 m.g.d. of 38 minutes.

Two sedimentation basins will have a total capacity of 875,000 gallons, giving a retention period at 6.0 m.g.d. of 3.5 hours.

Four filter units with a nominal capacity of 1.5 m.g.d. each will be constructed, a total of 6.0 m.g.d nominal capacity.

The filtered water basin will have a capacity of 1,250,000 gallons.

The filter plant will be constructed with the reaction basins along one side of the pipe gallery with the sedimentation basins beyond the reaction basins and away from the pipe gallery. The filtered water basin will form the other side of the pipe gallery together with the filter units which will set on top of the filtered water basin.

The pipe gallery is arranged so as to provide easy access to the various filter piping. Good flexibility is provided to the extent that part of the filter plant may be operated independently. The high lift pumps can take suction from the filter water basin or directly from the filters. An emergency arrangement is provided so that the high lift pumps can pump settled water to the Village in case of an unusual fire demand. This connection requires the connection of a fitting so that there is no danger of polluting the filtered water supply under normal operating conditions.

Due to the need for prevention of pollution of the bathing beach, all of the wash water from the filters and sludge from the sedimentation basins will be discharged into the sewerage system of the village. A wash water storage tank is provided to limit the flow of water into the sewerage system and to allow flexibility in the washing of filters. This tank will also allow dilution of the sludge from the sedimentation basins if necessary.

The plant will be provided with an office, laboratory and substantial space for the storage of chemicals. A garage is provided with space for two cars. A work or tool room is also included.

The dry chemicals will be applied through the use of four dry feed machines, providing for alum, lime and activated carbon. The piping is so arranged as to allow any machine to apply any chemical to all points of application. Chemicals may be applied to the raw water, settled water and filtered water and may be added between the two sedimentation basins for double coagulation.

It is planned to use the top of the filtered water basin, outside the building line, for tennis courts. For this reason a gravel fill will be made on top of the basin and a thin concrete false slab constructed over the gravel for the courts. The gravel layer was provided for insulation.

Included in the project is the construction of a 16-inch force main 1800 feet long which connects into four existing mains.

An existing 400,000 gallon elevated tank rides on the distribution system.

(Presented before the Illinois Section meeting, April 19, 1933.)

FACTORS AFFECTING THE SANITARY QUALITY OF DEEP WELL SUPPLIES

By F. G. Merckel

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Beginning with the notable report on "Sanitary Control in the Development of Ground Water Supplies," presented at the Conference of State Sanitary Engineers in April, 1925, an increasing number of writers have contributed to the literature on this subject, perhaps the latest paper of note being that by Klassen and Ferguson in the June, 1932, *Journal* "Sanitary Specifications and Construction for Water Wells."

In the last named paper, the authors remark, "With some the erroneous impression still prevails that the mere fact water is being pumped from a well gives definite assurance of its safety." The personal experiences of the writer and of others who have attempted to interest those responsible for municipal supplies from well sources in safeguarding these supplies confirms this statement. The fact that 1806 cases of water borne typhoid fever were traced to polluted well supplies in the United States during the past decade emphasizes the urgency of counteracting such an erroneous attitude. Add to this the fact that 371 out of 470 sources of public water supplies in Illinois are wells; that 65 percent of the 11,000 public water supplies in the United States are obtained from wells or springs and the Wisconsin State Board of Health estimate that there are 300,000 private wells in that state alone, the need becomes increasingly apparent.

A study of the many State Geological Survey Reports as well as of the Water Supply Papers of the United States Geological Survey reveals an astounding paucity of discussion of the sanitary quality of the many thousands of supplies reviewed. In fact, most writers of those papers when discussing "contamination" and "quality" have had in mind only such chemical contamination and quality as salinity and hardness. This point of view is not surprising when we consider the fact that the greater portion of the published geological data on ground water supplies appeared before bacterial examination of water became routine or before water sterilization was known or gen-

erally accepted. Such writers are therefore not to be blamed if, when confronted with a choice between a highly polluted surface water supply and a ground water supply of inevitably better history, they advocated the use of the latter. What has been lost sight of perhaps in the intervening years is that engineers have taken this surface water supply of *known* doubtful quality and have rectified it, whereas the original tradition as to the prima facia superiority of the well water supply without rectification has clung often with consequent neglect of even elementary precautions.

This last statement is not entirely true when applied to shallow well water supplies, particularly dug or driven wells. For some time there has been an increasing recognition of the unfitness of such supplies for public water supplies in closely populated areas unless treated as surface water supplies, which in effect they are in such areas.[1, 2, 3]

[1] Isaiah Bowman in U. S. G. S. Water Supply Paper no. 257 (Well Drilling Methods) says: "The driven well has the disadvantage of using surface seepage water which is likely to be contaminated."

[2] U. S. G. S. Water Supply Paper no. 258, pp. 57-65 (Protection of Shallow Wells in Sandy Deposits—M. L. Fuller): "Where a single source of pollution exists and only a small amount of polluting matter enters the ground, the contamination does not commonly extend beyond 150 feet. An open well at this distance would probably give no trouble and a driven well extending 15 feet or more below the water level is almost sure to be safe, providing the rate of movement of the ground water is normal and the pumping not severe. Where there are several sources of pollution and large amounts of polluting matter are introduced into the ground the contamination may extend in porous materials for some hundreds or even thousands of feet, especially when the underground waters move with considerable velocity. In such places open wells are out of the question, but driven wells carried 20 or 25 feet below the water level will usually afford safe water if not heavily pumped. If the wells are heavily pumped to obtain water for manufacturing purposes or for city supply the water table is usually lowered considerably, the polluted water is likewise sinking to a greater depth. Under such conditions a depth of 50 feet or more below the normal water table is none too much."

[3] U. S. G. S. Water Supply Paper no. 234, pp. 68-77 (Underground Waters—W. C. Mendenhall): "Contamination of water in shallow drift wells has been the cause of many typhoid epidemics in Connecticut as at Bristol, Easton, Glastonbury, Guilford, Madison, Middletown, Portland, Ridgefield, Stanford and Stafford. . . . . In Hartford, Waterbury and other cities analyses of water derived from sand and gravel deposits commonly indicate considerable sewage contamination, although the water is obtained at points 30 to 50 feet below the surface. In fact, it is doubtful if any well is safe in a thickly settled community. . . . . That contaminated water may circulate through considerable distances of sandstone is shown by the experience of Hartford, already described."

The most obvious hazards of both shallow and deep wells such as insufficient curbing or covering, pit drains to sewers, flooding, leaky suction pipes, contaminated storage reservoirs or receiving wells and many others of like character have been adequately covered in the literature, particularly in the "Report of the Committee on Sanitary Control of the Development of Ground Water Supplies" of the Conference of State Sanitary Engineers. For that reason they are not discussed in this paper, which attempts to concern itself with the more remote and perhaps more often ignored dangers.

In spite of the apparent greater danger of contamination of the shallow well, the deep well is of even greater concern to the sanitarian because it is the source of supply of larger communities and because its vulnerability is frequently masked.

THE IDEAL DEEP WELL SUPPLY

An ideal deep well supply should have specifications somewhat as follows:

A remote catchment area, an inclined strata of sufficient cross section tapping the catchment area, the catchment area to be large enough and the aquifer (water bearing formation) to be porous enough to insure adequate quantity at the point where the well is drilled. Preferably the catchment area should be at an elevation higher than the ground level at the well site to give adequate head to the supply.

Our specification so far, except as to the remoteness of the catchment area, has been concerned with quantity and pressure. It is further obvious that an impounded surface supply would have very similar requirements.

As regards the chemical quality of the water, it is desirable that the supply, in its travel from the catchment area to the well, does not traverse a rock formation from which iron, manganese and the salts of calcium and magnesium are too easily leached.

As regards the sanitary quality, it is necessary that any cross connection with polluted supplies be absent. This simple statement comprises, however, a wide field of possibilities including fissures in the rock formation itself as well as pot holes and caverns; that casings which penetrate through polluted strata remain indefinitely unperforated by corrosion or other causes; that annular spaces around such casings communicating with polluted veins remain permanently sealed; that the same shall be true of all other wells which have been drilled to the water bearing strata in the past or that will

be drilled in the future else these may become responsible for polluting an otherwise perfect well. It includes, furthermore, the perfect sealing off of all abandoned wells and the elimination of all drainage wells which may be the cause of undesirable short-circuiting.

It is not difficult to visualize that as our population density becomes greater and greater and as our country becomes older and virgin territories become scarcer, as individual wells fail and are abandoned and as ground water tables become lowered with increased tapping of the resources, such specifications become increasingly hard to meet.

Bowman[4] classifies the various rock formations as to their water bearing possibilities in this manner:

Sandstone—is the source of the best quality and quantity of ground water.

Limestone—The water in limestone occurs mainly in open channels and caverns and is likely to be polluted owing to the fact that much of the water in the underground streams in limestone has found its way downward through sinkholes and carries with it more or less of the surface wash.

Shale—Water from shale is usually poor in quantity and highly mineralized though seldom polluted.

Granite—Gneiss and Schist—holds water in joints—the joints are most common near the surface, usually within 200–300 feet of the surface. "The joints in crystalline rocks usually form complex systems of intersecting planes and polluted water may pass in a zigzag course until it finally reaches the well at a depth of many hundred feet."

At the outset, in the location and drilling of any given well, the driller is confronted first with the problem of finding an adequate quantity of water; secondly, with finding a usable quality of water, particularly from the standpoint of mineral content. The choice is confined in most cases to one or two possible water bearing strata and the question of balancing quantity, chemical quality, construction cost, pumpage cost and the life of the well are of such importance as to overshadow the question of possibility of bacterial pollution, particularly when such pollution is not immediately apparent.[5]

[4] U. S. G. S. Water Supply Paper no. 257 (Well Drilling Methods—Isaiah Bowman).

[5] The problem of cost is usually complicated by limited finances available on the part of the community involved or by the desire to make a large showing with the finances allotted for the project and finally by the red tape and competitive system of bidding enforced on works of public character. F.T. Thwaites remarks (Stratigraphy and Geological Structure of Northern Illinois, 1927—Illinois State G. S. Report of Investigation no. 13), "Well drilling is an art that requires experience and ingenuity; it is an operation in which,

The water bearing strata of most importance to the driller are those of limestone and sandstone. The limestone is subject to considerable risk of direct pollution. This statement is not made on purely academic considerations, but has, unfortunately, often been proven. At Springfield, Ohio, for example, in 1928 at the Ohio Steel Foundry Company a deep well in limestone which had apparently been safe for years, suddenly became polluted following a period of wet weather, caused 37 cases of typhoid fever with 4 deaths.

The Riverside, Illinois, outbreak in 1929 was probably due to underground leakage of the Des Plaines River through creviced limestone to the city wells. Over 1000 persons were affected by dysentery as a result of this pollution.

At Sturgeon Bay two typhoid epidemics in 1921 and 1922 occurred from the same well. Leakage from a sewer travelled through crevices in the limestone formation to a hospital well. The first outbreak of 21 typhoid cases with one death was followed seven months later by a more severe outbreak of 31 cases and 3 deaths,—the penalty for not having corrected conditions responsible for the original outbreak.

In fact, the literature teems with example after example of contamination and outbreaks of intestinal diseases, directly or indirectly attributable to pollution from limestone strata. Gorman says, ("Water Borne Outbreaks due to Pollution of Ground Water Supplies," delivered at the Sixth Annual Missouri Water and Sewage Conference).

"Were it possible for every health and water works official to see the condition of geological formation in which many of our limestone wells obtain water and to observe the changes, movement, character and elevation of ground water after rains and during drought periods, there would be an insistent demand for better protection of public water supplies from these sources.

"A fact which needs great emphasis is that water which flows in channels in limestone probably receives less natural purification than surface water. Especial attention is called to this fact because it is a popular belief that water is purified by passing a short distance underground. This belief is based on the study of the purification of water by its passage through sand or sandstone. In these materials the purification is mechanical and takes place because the openings in the grain are very small. In other words, in removing the objectionable matter the sand acts as a mechanical filter.

"Water that flows through channels in limestone receives no filtration and consequently but little natural purification. It is also effectually sealed from

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many times, the personal factor is more important than the quality of the machinery. The lowest bidder is often the most expensive in the long run."

the oxidizing effects of air and sunlight. The flow of water in limestone passages may be compared with its movement through ordinary water mains, the chances for purification are about equal in the two channels."[6]

Wells in sandstone have not been exempt from sub-surface contamination, but in most cases such pollution has occurred on account of auxiliary defects such as imperfect packers (Lockport, Ill.), abandoned wells (Fond du Lac, Wis.), fissures caused by dynamiting (Bad Axe, Mich.). The record of sandstone aquifers is on the whole infinitely better than that of limestone.

So much for the geological formation's relation to water quality. Extended comment on the admitted possibilities of contamination through fissures, joints, crevices, etc., in rocks of igneous origin is not justified in view of the limited quantities of water found in these strata, precluding their use in most cases as sources of public water supplies. However, such fissures, etc., in igneous rocks overlaying or communicating with water bearing strata may result in contamination.

CONSTRUCTION METHODS

How does the manner of constructing the deep wells affect their quality? W. H. Jeffery ("Deep Well Drilling"), speaking on oil and gas well drilling:

"The shutting off of water is the chief and most important purpose for which casing is used. . . . The water in stratified rocks presents many problems to the oil and gas operator, is the cause of much expense in drilling and when careless or unintelligent methods of shutting it off are employed, may be a menace not only to his own property, but to the properties of his neighbors.

"More attention has been given to the problem of casing of wells and to the conservation of oil and gas by correct well engineering methods by engineers and the U. S. Bureau of Mines and more has been written on these subjects than on any other phase of well drilling technology.

"Each oil field has its own peculiar requirements of casing. In the more than 100 oil fields of the United States there are as many different combinations of sizes and length of string."

This statement is of considerable interest. Drilling for, and producing oil, is a commercial proposition—a loss in production cannot be tolerated. Apparently a tremendous amount of engineering skill has gone into the solution of the problems involved and a great

[6] "Pollution of Underground Waters in Limestone," by George C. Matson, U. S. G. S. Water Supply Paper no. 258.

many recognized difficulties persist. Contrast this with Bowman's statement (U. S. G. S. W. S. P. no. 257):

"In water wells less care is exercised in packing, plugging and casing than in oil or gas wells, although the neglect may have serious sanitary results even if it is of less obvious economic importance."

He says further:

"The water in water wells may be contaminated and rendered unfit for use by allowing water from other sources than those yielding desirable supplies to enter the well. In drilling a well, water may be found at several levels, one of which yields unpalatable water and it is then desirable to separate the different water bearing beds and determine the source of the undesirable water. By the proper arrangement of pipes, packers and plugs each water bearing bed can be separated from the others, and examined and when the undesirable bed is located water from it can be excluded by casing it off."

The last sentence will bear examination from a sanitary viewpoint. The manner of examining for and eliminating chemically undesirable water bearing beds is readily apparent. How to do this bacteriologically, with constant contamination from soil and tools while the driller is holding up crew and rig is not so apparent. Even if such bacteriological examination during construction were feasible, what will happen once the well is heavily pumped and the draft on any particular strata becomes great, may be an entirely different story. Chemically this would have no significance, an increase in hardness of a grain or two is a matter of no great moment, but an increase even slight of bacterial flora, should they be of pathogenic nature, may be of tremendous significance.

If the shutting off of water in oil wells, where casing is used from the ground to the bottom of the well, is a difficult matter, how much more is this the case in water well drilling where a fully cased well is the exception, rather than the rule?

Thwaite says:

"An excellent shut-off may be made by cementing the bottom of each string of pipe. This is best done before the hole is drilled any deeper, but many drillers prefer to leave the casing until all drilling has been completed for fear that in a crooked hole the wire line will wear through the pipe. It is advisable to use cement thick enough to protect the well from contaminated waters even without any pipe; this should extend from the surface through all contaminated waters. At lower levels many prefer clay instead of cement since that does not prevent the withdrawal of rusted pipe."

But, the same author says:

"That casing suffers deterioration and decay and that it should be examined at intervals for resulting defects is, however, shown conclusively by its condition when it is withdrawn after having been in the earth only a short time."[7]

If we are to cement this casing in, in spite of the risk of the wire wearing through it, how are we going to draw, examine and renew it?

Many other problems confront us as we proceed with the construction of the well. For example. "It must be realized that holes of different sizes of pipe are rarely concentric and that lead seals between different sizes of pipe makes joints of indifferent quality. It is far better to follow oil well practice and have the last string of pipe extend to the surface."[8]

Caving is a cause of difficulty in maintaining seals against polluted strata. Rock too soft to support the weight of the casing at the desired shutoff level adds its problem. Shooting of wells to increase yield often causes permanent caving conditions.

Crooked holes (due in some cases to inclined fissures) cause trouble in sealing, shut off, casing and casing renewals.

These construction handicaps mitigate against production of a water of high sanitary quality to the extent that they make difficult or hazardous the keeping out of possible polluting water. Public water supplies are located in practically all instances at points where due to population densities pollution is an ever present factor. The hazards of construction are therefore, very real ones and the types outlined above are ones difficult to avoid.

OPERATION

When the well has been completed and placed in operation, additional factors must be watched, notably deterioration of the well casing and excessive drain and draw down of the water table.

"The life of a casing cannot be definitely predicted, the rate of its decay depending on the special conditions in each well. Casing withdrawn from some wells fifteen to twenty years old has been found to be in fairly good condition except at the joints, though as a rule at this age it is too badly corroded to be withdrawn at all."[7]

"At Dallas, Texas, the writer (Bowman) observed holes the size

[7] Bowman, U. S. G. S. Water Supply Paper no. 257 "Well Drilling Methods."

[8] Thwaite, Illinois State Geological Survey Report of Investigations no. 13.

of a cent in casing withdrawn after having been in the earth but one year."[9]

Wisconsin Geological Survey Bulletin no. 7 says, "In some wells the iron pipes have been corroded within a period of two years."

Even, therefore, if we succeed in shutting out undesirable or polluting water during the well construction, we have no guarantees that we will keep such contamination out as casing corrosion may set in immediately.

Water from a contaminated soil found entrance through the upper rusted casing of a well in Waterloo, Iowa and caused a typhoid fever outbreak.

"At Cedar Rapids a wrought iron casing was corroded to a perforated shell in about five years. At Grinnell the water of well No. 1, containing 2000 parts per million of mineral matter, rusted through the casing in about eight years."[10]

"The rusting of casings must be regarded as one of the most serious difficulties in maintaining a supply of artesian water."[11]

"At Fond du Lac it has been found by the Superintendent of the County Asylum that ordinary iron pipes (in wells) last about fifteen years. . . . in many places only half as many years."[12]

At Kenosha, Wisconsin "Wells are nearly abandoned. The pipes have partly rusted and there is considerable leakage."[12]

Casing corrosion is a factor difficult to control under practical operating conditions—few of the many thousands of wells in operation have their casings protected by cement grouting around them—in fact, all too few have sufficient casing. Many wells in use today are from twenty to forty years old—the first wells for instance, in Illinois on public water supplies date back to 1868 (Peoria, Springfield) and the greatest activity in well construction in that State was in the 1890s. With a natural expectancy of 15 to 20 years for ordinary iron pipe in soil, many of these casings must be in a precarious condition.

That the water table will drop on continued and heavy pumpage of a well is too well known to require comment. That such drop in water table elevation is accompanied by increased risk of surface

[9] U. S. G. S. Water Supply Paper no. 160, pp. 92–95 "Problems of Water Contamination." Bowman.

[10] Iowa Geological Survey, Vol. 33.

[11] Iowa Geological Survey, Vol. 21, 1911.

[12] Wisconsin Geological and Natural History Survey Bul. no. 35.

pollution is not always apparent. This situation is aptly illustrated by an example given by L. C. Glenn[13] where, at a number of places in South Carolina, water has been obtained by sinking deep wells uncased below bed rock into the fissured crystalline rocks that lie below the soil layer. He says:

"The deeply drilled hole has cut across enough such fissures to obtain a large supply of water for the pumps, yet this water is derived not from a great depth nor from a long distance, but from the great body of ground water near the surface and immediately around the well. If the fissures in the solid rock are very numerous the area supplying the water is an inverted cone whose apex is the lowest point of entry in the borehole and whose base is a large circular area of the ground water surface around the well mouth, and only one, two or three score feet beneath the surface of the town, with its innumerable sources of pollution of every kind. If the well is supplied by only one fissure or by a few fissures, they must be correspondingly large and furnish more direct and easy lines of flow to the well. They may extend almost uninterrupted nearly up to the surface and furnish a direct channel for surface waters to enter the well.

"The amount of water that can be pumped from such wells is usually large and analyses made when the well is first completed have shown a good, pure, clear, and usually soft water, well adapted to almost any purpose. But while at first such deep wells may be protected from contamination by the surface-soil layer, which acts as a filter, and the water may be pure and wholesome, yet they are very apt sooner or later to become contaminated. Constant pumping from them soon causes the indrafts supplying the water to open better channels through the fissures and allows more direct ingress of waters from the surface. These channels sooner or later are able to carry surface contamination directly into the well. Such deep wells are therefore at best open to grave suspicion and constantly need careful watching. This need is all the more necessary because of the fancied security given by the very favorable analysis of the water when first used. Such careful watching will usually result in the condemnation and closing of the well in a few years, especially if the town be of considerable size, so that there is furnished to the soil a large amount of contaminating organic material. A well of this kind drilled over 2,000 feet deep in Atlanta, Ga., some years ago, which at first furnished a large amount of acceptable water, was ordered closed by the Board of Health after a few years use because of sewage pollution."

ABANDONED WELLS

Unless brought under strictest supervision and control abandoned wells will be an increasing and major threat to both the quantity and quality of ground water supplies. Many states have hundreds of

[13] "Underground Water of South Carolina." U. S. G. S. Water Supply Paper no. 114.

abandoned wells on their public supplies and probably several times as many abandoned private well supplies. No state has adequate legislation inspection and control over them.

Illinois State Water Survey Bulletin no. 21 mentions 164 cities in the State of Illinois which have abandoned one or more wells. It is estimated that over 400 wells on public water supplies alone have been abandoned. In only two or three places in this report is mention made of any of these wells being plugged or filled. Unless such abandoned wells are grouted up they must obviously represent a hazard either through accidental contamination as occurred at Fond du Lac or through corrosion of their old casings followed by direct access of the polluted waters to the lower strata—the very thing which these casings were originally meant to prevent. If this is the situation on public supplies supposedly controlled and supervised by public health authorities how much more chaotic must it be on private supplies. It is a situation furthermore which places the operator of a properly constructed and operated well at the mercy of any careless well owner anywhere within a considerable radius.

This situation we believe is as true of Wisconsin as of any other State. For example, in 1907 it is reported that in Fond du Lac "all the old wells reported by Chamberlain in the Geology of Wisconsin 1873–77 have been abandoned." Cities such as Oshkosh, Kenosha, Racine, Appleton have abandoned wells for surface supplies. It is safe to guess that there are hundreds of abandoned wells in Wisconsin. At Watertown "over 50 wells have been sunk into the St. Peter sandstone and furnish flowing wells (1907). Many of these have been neglected. . . . The abandoned wells, greatly reduce the head of the St. Peter horizon. . . . If these old wells could be plugged so as to prevent the escape of the waters into the drift at lower depths. . . ." Not only are we aware of their abandonment, but we know also that in the major number of them casings are corroded, seals imperfect and leakage is taking place.

The abandoned well is not only subject to accidental contamination but unfortunately is often deliberately made into a sewage or drainage well for the disposal of undesirable wastes ranging from rubbish to sewage and industrial wastes. Such cases are on record in Wisconsin as well as in most other states. What more convenient method of disposing of noisome creamery wastes, for example, especially where riparian owners downstream are protesting violently and threatening suits.

DRAINAGE WELLS

This brings us to the subject of the deliberate construction of wells for drainage purposes—a practice more widely spread than would be first supposed.

Myron L. Fuller (W. S. P. no. 258, U.S.G.S.) informs us that:

"Drainage into sandstones is said to have been successful in Michigan and several wells in St. Paul and Minneapolis carry refuse into the porous St. Peter sandstone." . . . . "Wells sunk into limestone have been used for the disposal of sewage in Kentucky, Georgia, Florida and possibly other states." . . . . "The drainage of ponds in Michigan by wells has already been noted (p. 16). In Minnesota very similar results have been obtained at several places and in Wisconsin, Indiana and other states where the surface deposits are of a similar type drainage wells have been successfully employed. . . . In Virginia, Kentucky, Tennessee, Indiana and other states in which limestone occurs it is a common practice, it is said, to dig or drill for the purpose of carrying the water out of sinks into the underlying limestone. . . . Little is said by owners of borings into which industrial wastes are being turned, but it is probable that such drainage is being practiced in Minneapolis and St. Paul and in certain limestone areas in Georgia and Florida. Creamery waste products are disposed of in this way in some country towns. . . . Among the cities in which there are public or private sewage wells are Georgetown, Ky. and Orlando, Ocola, Live Oak, Gainesville and Lake City, Fla."

A number of years ago the writer was informed by those in charge of the water supply at Mineral Point, Wisconsin that the practice of disposal of sewage by means of wells was common in that city on account of the lack and expense of, sewerage due to the location of the city on rock outcroppage.

EXAMINATION OF GROUND WATERS

It is the practice in most states to make bacterial analyses of ground water supplies at comparatively infrequent occasions, once or twice a year, unless a purification process such as iron removal, or chlorination, is used on the supply when more frequent samples—as high as one a week—are taken. Increasing awareness of the hazards surrounding well supplies would indicate more frequent examination of unpurified ground waters.

In Iowa there are in excess of 500 municipal wells classifiable as deep wells and ranging in depth from 100 to 2122 feet. The Iowa Geological Survey (1928, Vol. 33.) reports "The fact that in 63 percent of the towns of Iowa using deep wells in part or whole for water supply, contamination has been proved on *two* or more occasions

shows conclusively that deep wells, like shallow wells, may be polluted by surface water." Over 300 of Iowa's wells, therefore, have shown such surface pollution.

In Kansas, less than 1 percent of the ground water supplies of that state had perfect records for a period of eight or nine years.

Certainly these facts would indicate that ground water contamination is not a sporadic occasional incident, but something widespread, common and not to be ignored.

CONCLUSIONS

A well, because it taps a source of supply in common with other wells and because its "watershed" and "storage reservoir" are not readily open to inspection is peculiarly susceptible to contamination beyond the control of its operator.

To minimize the hazards surrounding it, it is advocated that:

The definition of public water supplies be widened to include all wells other than shallow private wells of strictly local influence and that these be placed under public health supervision, since it has been shown that any such wells in the course of their construction, operation and abandonment may become a menace to others.

Such public health supervision be reinforced by adequate legislation and personnel to insure more than nominal inspection and control.

There be more frequent bacterial and sanitary analyses and inspections of public well supplies as an advisable safeguard.

Disinfection of well water supplies furnishing water to anyone but the owner thereof and his immediate family, is a justifiable, sensible and necessary safeguard sooner or later in the life of any well and that it is good sense to take such precautionary action before, rather than after, contamination is discovered.

(Presented before the Wisconsin Section meeting, October 12, 1932.)

THE CRISIS IN STREAM GAGING IN NORTH CAROLINA

By H. D. Panton

(Chief Engineer, Department of Conservation and Development, Raleigh, N. C.)

I regret to choose for my subject such a title as "The Crisis in Stream Gaging in North Carolina," for I am sure that all of us have thought and hoped that after thirteen years of effort we had developed in North Carolina a reasonably comprehensive program of stream gaging which would be still further expanded until we were collecting stream flow records from all streams of public and economic interest in the State.

The Water Resources and Engineering Division of the North Carolina Department of Conservation and Development was created in 1920. At that time there were in operation in the State only 8 gaging; stations, not one of which was equipped with a recording instrument the funds expended on this work for that year by both State and Federal agencies amounting to about $4,350. By 1931 there were in operation in the State 77 gaging stations, 48 being equipped with recording instruments; and the money spent for this work from all sources during that year amounted to $34,310.00, which was, however $5,131.00 less than had been spent in 1930, that year seeing the peak of expenditures for stream gaging. Since that time each year has seen a shrinkage in the money available for carrying on this work and a consequent curtailment of the scope of the work that could be undertaken. At the end of the last fiscal year, June 30, 1933, the number of gaging stations in operation had been reduced to 69, of which 12 were entirely supported by Federal funds without state cooperative aid and 57 were maintained cooperatively by the N. C. Department of Conservation and Development and the United States Geological Survey. Financial cooperation in the maintenance and operation of several of these stations was also received from interested municipalities, power companies, and industries; but such cooperation has declined greatly in the past several years.

The 1933 session of the State Legislature in its efforts to accomplish the laudable purpose of balancing the State's budget for the

current biennium drastically reduced the appropriation for the Department of Conservation and Development with the result that we now face in the Water Resources Division a crisis in the matter of funds with which to carry on our stream gaging work. We have available from State funds only some $4,500.00 for the current year, $3,000.00 of which is for cooperative work with the United States Geological Survey.

The Geological Survey, due to restrictions in its own funds, has not been able to fully match dollar for dollar the State funds available and stands definitely committed to spend on coöperative stream gaging in North Carolina only $2,250.00 during the current year. Including the funds for stations entirely supported by Federal agencies the Asheville office of the Survey faces the critical situation of undertaking to carry on with a reduction of over fifty percent in its operating funds. This situation is somewhat ameliorated by the fact that $14,000.00 of Public Works Administration funds have been allotted to the North Carolina District for maintenance and improvement of existing gaging stations, and the Tennessee Valley Authority has stated its willingness to make some contribution towards the support of the 23 gaging stations located in the western part of the State in the watershed of the Tennessee Basin, but have made no definite allotment so far for this purpose.

The Asheville office of the Geological Survey has in active operation at the present time 68 gaging stations, and Mr. Burchard, the District Engineer, by careful economy hopes to be able to continue this number of stations in service for the current water year ending next September 30; provided sufficient assistance is received from the Tennesseee Valley Authority. Without their help the number of stations will shortly have to be reduced to 48; and the records from 18 of these will be filed for future computation when funds for this purpose are available.

I know that you are individually and collectively through this Association interested in stream gaging in North Carolina; and that through the North Carolina Section you have always supported and used your influence to further this work. About the vital need for this work there can be no question if additional water supplies for our cities and towns are to be efficiently planned as future growth and development causes present supplies to be outgrown. The present economic depression will not last indefinitely, and in order that you may have the needed stream flow data when the com-

munities of our State have once more resumed their forward advance and additional water supples are needed, we must by some means manage to continue our gaging program and to keep, if it can possibly be done, all of our present 67 gaging stations in service. Indeed, to fulfill adequately our function we should expand the scope of our work, but with the existing situation such as it is we cannot hope to do this until the return of more prosperous times. We now face the prospect of possibly having to greatly curtail our present program before the end of the current biennium.

We have spent on stream gaging in North Carolina a total of approximately $300,000.00 in the past thirteen years, and the records from the majority of the gaging stations still cover too short a period to be of much value until we have collected them for several years longer. Records covering a period of less than 10 years are of doubtful value, and the records of a stream should cover a period of 30 years to present a reasonably true picture of stream flow characteristics. Thus stream gaging to be effective must be continuous and carried on as a permanent study. Of the 67 gaging stations now in service we find:

5 with a record of over 30 years
2 with a record of 20–29 years
8 with a record of 10–19 years
38 with a record of 5–9 years
14 with a record of less than 5 years

From the above it is evident that a drastic curtailment of our stream gaging activities will not only greatly reduce the value of this work, but will cause much of the money and effort that has gone into it in the past to be wasted, since many of the records on stations that would be discontinued cover too short a period of years to be of much value. But with each year we can keep such stations in service the value of the record increases. We have 38 gaging stations which have been in service over 5 but less than 10 years, and only 15 in the entire State with records of 10 years or more. Should stream gaging activities in North Carolina be discontinued altogether, a not impossible outcome of the present situation, these fifteen records would be the only ones of sufficient duration to be of real value.

Such being our situation in regard to stream gaging we feel this matter should be called to your attention at this time; and we wish to request that you assist us in two ways—both as the individual

representatives of the water departments of your respective communities, and collectively through this Association. First, we would like to have you help us by using your influence with your representatives in the next State Legislature to secure from the Legislature adequate financial support for stream gaging; and second, through your Representatives in Congress to advocate an increase of the funds appropriated to the United States Geological Survey for coöperative stream gaging.

As the U. S. Geological Survey is limited by statute to a 50-50 financial coöperation with the states on stream gaging work, our present effort should be to lay the necessary ground work for securing an adequate appropriation for this work from the next session of the State Legislature. Until then it will be necessary to get along as best we can with the funds available, keeping as many stations as possible in operation. Before discontinuing a station we will take up with the communities and private interests, if any, who we feel may have use for the records of that station the matter of local coöperation to tide over the present emergency. When such local coöperation is secured we shall undertake to keep the station in service.

(Presented before the North Carolina Section meeting, November 13, 1933.)

COLLECTING BILLS IN DOVER, NEW JERSEY

By George F. Steffany

(President, Board of Water Commissioners, Dover, N. J.)

No matter what financial stress may strike a community, four cardinal branches of government must be maintained. These are the water works system, the fire department, the police department and the health department.

Without water the fire department could not function and without a potable water supply there would be a general exodus of the populace, after which the police and health departments could fold up their tents, like the Arabs, and silently steal away.

The public water supply system is, therefore, the key department of community life and nothing can be countenanced that will tend to cripple its financial structure, for on this the proper functioning of the water department hinges. All bills must be collected when due. We have plenty of troubles with bursting water mains, leaking valves, fire hydrants and reservoirs, short circuits in motors, stopped or frozen meters, filtration, chlorination, without adding arrears in water rents.

Whether what Dover, N. J., with its 10,000 population and 2500 water meters, has accomplished is applicable to your problems, only you can judge.

Less than $150.00 is due on uncollected water rents and these are on vacant properties where the water is shut off. No other person owes his water bill to the department for current water supply.

We have not changed our policy in these years of depression.

All payments must be made to our office within 30 days to receive the 10 percent discount. Thirty days further grace is given before payment becomes delinquent. Then we start collecting.

The telephone is used and the debtor given until 10 o'clock the next day to pay his bill or have his water shut off. He knows we mean just what we say and then we charge $1.00 for the trouble of shutting off the water and $1.00 for cutting it on. These charges are made and collected before the water service is restored.

Perhaps someone may think that this is a high-handed policy and an undue exercise of power, but it is not, because it is necessary.

We find there are very few people who are arbitrary and resistant and we also know that were a precedent of leniency established for these few, collections could not be made from those who would capitalize on that precedent.

In our community, everyone knows the water bill must be paid when due or the water will be shut off at their expense. Somehow or other we do not have to do it. The money is collected either by telephones or the superintendent or his deputy at the home.

This is just hard-hearted business enforced by the might of the shut-off valve key.

Someone is sure to be thinking you cannot get blood out of a stone and you cannot collect even a very small sum, if the people have no money and can get none. You are right.

But every water works department needs common laborers in its everyday operation and maintenance.

We have no common laborers in our employ. We hire for that purpose those who want to work out their water bill. Usually two days will suffice. They are paid in cash. Then they pay the water bill and usually have a few dollars left over. This is just the same idea as the Civil Works Act, except that we do not make work. Our employment is for work that must be done for proper maintenance or extension purposes. A man can get this employment almost anytime, even to leaving his pay check in escrow to pay a water bill not yet rendered.

We have no poor people in distress because of water rents. No poor people owe us money; we are both happy and safe. This is not new with us. We have for years tried to help in this way, but at the present time it is much more extensive, yet we do not find that there is apt to be any harmful aftermath to cope with. Even though the depression recovery is slow, the supply of labor is not greater than our demands. A water works commission is just plain lucky to be in the unique position to require so much common labor. It makes collections possible and also assures the continuance of that necessary service, the public water supply.

(Presented before the New York Section meeting, December 28, 1933.)

DETERMINING THE QUALITY OF ZEOLITES

By Howard L. Tiger

(Technical Manager, The Permutit Company, N. Y.)

Zeolites have been used for over 25 years in this country and abroad for water softening. During this period, the application of this method of water softening has grown to be an industry of world-wide scope. In addition to their broad use for softening industrial waters for laundry, textile, boiler feed and canning purposes, etc., zeolites have established a definite place in the field of water softening for general use, such as is required in the home, the hospital, the office building and the hotel. This has been reflected in the steady increase in the number of installations made for these purposes until they now total well over 150,000. More recently, the tendency to use zeolites for general water softening purposes has made itself evident in the increasing number of installations made by municipalities in order to give entire communities the benefits of zeolite softened water.

The fundamental principles of the base exchange process are based on the following well known reactions:—

Softening

$$\text{Na zeolite} + \left.\begin{matrix}\text{Ca}\\\text{Mg}\end{matrix}\right\}\text{salts} = \left.\begin{matrix}\text{Ca}\\\text{Mg}\end{matrix}\right\}\text{zeolite} + \text{Na salts}$$

Regeneration

$$\left.\begin{matrix}\text{Ca}\\\text{Mg}\end{matrix}\right\}\text{zeolite} + \text{NaCl} = \left\{\begin{matrix}\text{CaCl}_2\\\text{MgCl}_2\end{matrix}\right. + \text{Na zeolite}$$

It is apparent that an industry which has achieved such substantial growth and broad application in the course of a few decades, must be founded on a process which has a number of great advantages over any previously known method of water softening. This phase of the subject has been thoroughly discussed in the technical literature. In the present paper, therefore, we shall merely take it for granted that the use of and need for zeolites in the field of water softening are well established and that it is therefore highly desirable to have further information on these substances.

NEED FOR DEFINITE INFORMATION ON ZEOLITE QUALITY

The growth in the industry has been accompanied by a great number of developments in the equipment utilized for the purpose, as well as in the zeolites themselves, which are the very heart of such equipment. There are, at present, at least three general types of zeolites on the American market and well over twenty distinct products which fall within these general groups. Strange to say, however, very little has been said or done to guide the user in determining the general type of zeolite to be preferred for his particular application or in deciding on the relative merits of the various products so that he can choose the best for his purpose. It is the object of this paper to offer some definite suggestions along these lines based on experience with various types of zeolites and on an effort to develop standard test methods for determining the quality of a zeolite.

This need for more specific knowledge is not confined, however, to the users of zeolite softening equipment. Without such specific knowledge we cannot expect to maintain precise manufacturing control on a material like zeolite, which cannot be judged by its superficial appearance, but must be judged by its performance. Nor can we expect the research worker to develop improved zeolites unless he has the tools whereby he can measure the quality of the product he has produced. The interests of the user in reaching a decision, the manufacturer in producing a reliable product, and the research man in developing improved products, are all, therefore, as one in the need for a reliable means of measuring zeolite quality.

WHAT IS QUALITY IN A ZEOLITE?

Since the main function of a zeolite is to remove the hardness by exchanging its sodium base for the calcium and magnesium salts in the water, there has been a tendency on the part of many to consider exchange value the sole criterion of quality. No idea could be more misleading and this is borne out by the fact that Greensand Zeolites (which as a class possess comparatively low exchange values) have come to be used in the vast majority of industrial zeolite softeners. What then are the criteria by which a zeolite should be judged? For convenience, these important properties of zeolites may be described under two general headings: physical and chemical characteristics. However, before considering these particular characteristics it is well to classify the various types of zeolites into groups so as to facilitate the study of any particular member of a group.

CLASSIFICATION OF ZEOLITES AND PROPERTIES OF GROUPS

The term "zeolite" is used in its broadest sense so as to include all substances that possess the property of base exchange and can be employed as a medium for removing the hardness or calcium and magnesium from water by replacing these elements with a corresponding amount of sodium base of the softening medium. Strictly speaking, some of the substances which may be so employed are not zeolites in the mineralogical sense, but usage in this connection has now clearly broadened the meaning of "Zeolites" to include such base exchange substances.

1. Glauconite (or greensand) group

These zeolites are greenish black, kidney bean shaped granules weighing about 80–90 pounds per cubic foot (dry basis). The exchange value of this group is about 2700–3800 grains $CaCO_3$ per cubic foot when using 1.4 pounds of salt per cubic foot in the regeneration. They are principally characterized by a strong resistance to aggressive attack and a comparatively low porosity. If overrun, these materials can be readily brought back to their original condition in a few regenerations.

2. Precipitated synthetic group (sometimes called wet process synthetic)

In general these materials have the rather high operating exchange value of about 6,000 to 20,000 grains per cubic foot on 4 to 8 pounds of salt per cubic foot. However, these higher values cannot be continuously obtained in practice. Due to this higher exchange value, the equipment required for removing a given amount of hardness is smaller than in the case of the glauconite group. This is particularly advantageous for household softeners where the small size of the units permits proper backwashing with the comparatively low flow rates usually available in households. Another advantage is that the grains of these synthetic zeolites may be made fairly large so that the head loss in downflow softening is relatively low.

On the other hand, these zeolites are sensitive to attack, particularly by waters of an aggressive nature, that is, waters which contain less than about 10 grains of hardness or are very low in SiO_2 or which have a pH value less than about 6.8. Being of a very porous nature, these zeolites are also sensitive to clogging by suspended matter, organic matter or iron. Furthermore, if they are overrun, it requires a great number of regenerations to bring them back to their original condition.

3. Fused synthetic group

These materials have operating exchange values of about 4,000 to 8,000 grains per cubic foot on 4 pounds of salt per cubic foot. This was the first type of zeolite used on a broad scale for commercial water softening. However, this class of material is not being sold extensively at the present time.

4. Clay group

These are manufactured principally from certain clays found in the Dakotas (bentonites, etc.). The clay is dried, granulated and baked, treated with alkali, washed, granulated and screened. The operating exchange value of these materials is about 5,000 to 8,000 grains per cubic foot on 4 pounds of salt per cubic foot. At the same time, they are porous and consequently sensitive to turbidity, etc., as well as to the attack of aggressive waters in the same manner as the precipitated synthetic materials. Thus these materials do not possess either the advantages of high exchange power (as do the precipitated synthetic group), or of high resistance (as do the greensand or glauconite type).

5. Miscellaneous group

This includes base exchange materials which can be manufactured from trass, slag, lignite and other materials. All of these materials have low exchange capacities as well as other disadvantages and, therefore, are not used for commercial water softening at the present time.

In view of the facts outlined above Group 1, Glauconite Type and Group 2, Precipitated Synthetic Type are used to cover practically the entire range of requirements for various types of waters.

It is difficult to lay down general principles to be followed in the choice of the proper group, since so many specific conditions may influence the precise decision required in each case. But there are a few general guide-posts which may be followed under most circumstances.

1. Where the volume of zeolite involved is small and the intensity of the softening service is relatively low, so that ruggedness and long life are not so important, *precipitated synthetic zeolite* may be used to advantage. This is particularly true in the household field where the average softener is designed to regenerate only about once a week, whereas the average industrial softener is regenerated about once or

twice a day. This means that for the same hardness of water each cubic foot of material softens about 6 to 12 times as much water per year in an industrial application as it does in a household application. Thus, if under certain conditions we would expect a given synthetic material to show signs of falling off in capacity after about 1 year on a certain industrial application with single daily softening cycle, it is safe to assume that full efficiency would be maintained for about six years on a similar water for a household application.

2. In most industrial installations small amounts of turbidity or alum or iron or a combination of these impurities frequently occur in the water to be softened. This is particularly true of surface supplies which constitute a large majority of our municipal waters. Also such treated waters (either after alum coagulation or lime-iron coagulation or lime-soda softening) are known from experience to be rather aggressive to zeolites. In most of these cases, therefore, the more rugged, properly stabilized zeolites of the *glauconite group are to be preferred.* This material is known to make an excellent filtering medium for such small amounts of turbidity, alum and iron. Moreover its ruggedness makes it capable of withstanding (without permanent damage) overrunning and under-regeneration frequently experienced in such installations.

3. On the other hand, the relatively low capacity of the glauconites may make *synthetic zeolite* preferable in cases where space is at a premium. This is particularly true on hard, clear, non-aggressive waters where synthetic zeolites may be used without danger and where it is possible to realize their additional advantage of requiring less rinse water. It is usually considered that for these materials the pH value of the water should not be less than 6.8 (this corresponds to a methyl orange alkalinity to free CO_2 ratio higher than 3). Also it has been found desirable to have a hardness exceeding 10 grains per gallon in the water to be softened by these materials.

4. With iron-bearing (chalybeate) well waters it is possible to obtain satisfactory removal of the ferrous iron together with the hardness by the use of the *glauconite type of zeolite.* Experience shows that this type functions very satisfactorily in this respect and that with properly designed and operated equipment the material stands up indefinitely.

With these facts regarding the various general groups in mind it is possible to study the physical and chemical characteristics which may be used as a basis for deciding on the particular material to be employed in any given case.

PHYSICAL CHARACTERISTICS

1. *Appearance.* Beyond indicating the general group or class to which a zeolite belongs, the appearance does not usually lead to definite conclusions even if the examination is made under the microscope. However, such an examination may give some idea of undesirable foreign matter that may be present with the zeolite granules.

2. *Feel.* This may sometimes be used as a rough guide to indicate whether the physical structure of a material has broken down in service. After extended use, under certain conditions, some zeolites may become more or less soft and gelatinous so that they can readily be crushed to a pulp between the fingers. But as an index to the physical strength of a new material, this quality of feel cannot be relied upon. In order to check this point seven different types of new greensand zeolites were recently subjected to a "blindfold test" by six men accustomed to working with zeolites. When the results of all were tabulated, they disclosed little or no agreement on the various materials.

3. *Screen analysis.* In downflow softeners, too fine a grain size increases the head loss during softening as well as the zeolite loss in backwashing. In upflow softening fine grain size may result in contamination of the effluent as well as in loss of zeolite.

On the other hand, the finer the grain size of a given material, the higher is its exchange value. This fact frequently results in a tendency to use fine grained material and thereby obtain higher exchange capacities. However, such a procedure obviously sacrifices ultimate satisfaction for a minor temporary advantage.

Much has been said of effective size in describing the grain size of zeolites and more particularly in connection with those zeolites produced from greensand. This term has been carried over from sand filter practice in which effective size means that size than which 10 percent of the material is finer. This figure is usually employed in conjunction with the term uniformity coefficient which is supposed to give an idea of the uniformity of the material. During the recent past, however, it has been felt that these terms may not adequately describe the quality of a sand for filtration purposes and therefore these terms are being supplemented by more complete screen analysis figures.

With respect to zeolites, these terms have even less significance than in the case of filter sand. Here the desirable condition is to have a product that consists of fairly uniform granules which contain

a minimum amount of fines. The best way to describe the grain size of a zeolite is to state its screen analysis in either tabular or graphic form. It is, of course, essential that such analyses be made by definite standard methods with the use of a specified standard screen scale.

4. *Weight per unit volume.* This factor is of particular significance in the case of the wet process precipitated sythetic zeolites which are placed on the market at a weight of about 50 pounds per cubic foot. This figure, however, gives no true indication of the actual weight of zeolite per cubic foot because the moisture content in these zeolites generally varies between wide limits. A certain amount of moisture is required for the proper preservation of the material to avoid powdering which might otherwise occur from the rubbing of the bone-dry granules in transit, but to state the weight per unit volume without stating the moisture content is meaningless. In other words, in order to obtain a true idea of the weight of zeolite per unit volume, this weight should be expressed on the dry basis, which means the weight of the zeolite when dried at 105°C.

Other things being equal, it is preferable to have a synthetic zeolite which contains a higher weight of true zeolite per unit volume because this gives an indication that the wall thicknesses in the granule itself are heavier and that, therefore, there is less danger of the granule breaking down as a result of wear and tear in service.

On the other hand, the greensand zeolites usually weigh about 80 to 90 pounds per cubic foot and the minor differences in the weights of these materials do not appear to have any particular significance.

5. *Porosity.* The general characteristics of the several classes of zeolites in this respect are well known. Small differences in this connection are not of much importance, but if one material is much more porous than another, it is evident that the more porous material will be more sensitive to clogging by small amounts of turbidity, iron, etc., which may be present in the water to be softened.

6. *Resistance to abrasion.* By this is meant the resistance to physical wear and tear only. It covers such qualities as toughness, brittleness and hardness. A material weak in this respect will be worn down by the abrasive effect of the friction of the water on the granules and the granules on each other.

In the case of greensand zeolites the resistance to this abrasive action depends upon the effectiveness of the stabilization which was applied to the raw greensand.

CHEMICAL CHARACTERISTICS

1. *Chemical composition.* In the case of the greensand zeolites, little information is disclosed by the chemical composition since the various greensand zeolites on the market have essentially similar compositions. Here the quality of the finished product is determined by the quality of the raw material and the method of processing, neither of which is clearly reflected by a chemical analysis.

However, with the precipitated synthetic zeolites, the chemical composition gives a clue with respect to two characteristics: (*a*) resistance to aggressive attack and (*b*) exchange value. In this class of zeolites, the composition is usually expressed as the ratio of $Na_2O:Al_2O_3:SiO_2$. Thus in speaking of a 1:1:6 material we mean a zeolite having the composition 1 mole Na_2O:1 mole Al_2O_3:6 moles SiO_2. Most of the variations in the composition of zeolites on the market occur in the SiO_2 group and most of these zeolites fall within the range of 1:1:4 to 1:1:15.

In general, other things being equal, increase in the SiO_2 content decreases the exchange value of the material, but increases its resistance to aggressive attack of waters.

2. *Resistance to aggressive attack.* This represents the rate at which the zeolite is dissolved by a given water or solution.

Other things being equal, it is more desirable to have a material which resists the dissolving action of water most effectively. And in this respect it is frequently advisable to sacrifice some exchange value in order to obtain a hardier material. A material that is relatively low in aggressive resistance will in time have some portions dissolved out of the grain (particularly with more aggressive waters), after which it will soften, gelatinize, disintegrate and waste away. Furthermore, as this deterioration proceeds the exchange value drops.

Since such deterioration of material usually requires an extended period in service, it has hitherto been impossible to recognize this weakness readily. For this purpose there have been developed certain accelerated test methods.

3. *Exchange value.* In this connection much confusion has resulted from a lack of understanding of the principles involved and a lack of recognition of the influence of the various factors on the results. To the untrained individual the determination of the exchange value of a zeolite appears simple; but actually such a determination requires careful control of a large number of factors in order to obtain reliable results. Variations in any one or more of

these factors may cause radical variations in the results. And by the same token the expression of these results without stating the control factors, make the results meaningless.

For a clear understanding of the results collected from widely different sources, there must be some agreement on the method of expressing the results, i.e., whether exchange value should be expressed per unit weight or per unit volume of the zeolite. Since the softener is designed with volume in mind, and since the weights of zeolites per unit volume vary widely even among materials of the same class, it seems logical to express the results on the basis of unit volume. This gives a common denominator over which all zeolites can be compared. And the expression of exchange value in terms of grains of hardness as $CaCO_3$ removed per cubic foot of zeolite is a convenient and satisfactory unit of measurement.

In order to understand exchange value more clearly, let us consider some fundamental principles. First, the total exchange value of a zeolite may be conveniently divided into three parts which may best be described graphically (fig. 1). From this diagram it is apparent that the total or ultimate exchange value (UEV) is never utilized in practice. Only a portion of this total capacity, which may be termed operating exchange value (OEV) is actually used in softening water and this portion is designated on the diagram as PV, which is always considerably smaller than RE, i.e., the ultimate exchange value (UEV).

Referring further to figure 1, it should be noted that points P and V are not fixed even for a given zeolite. Their location with respect to zero exhaustion and complete exhaustion depend upon a number of factors. For example, where salt is very cheap, P may be lowered to obtain more capacity per unit volume of zeolite and thereby decrease the initial cost correspondingly. On the other hand, where salt is expensive it may be desirable to raise P to that point where the maximum hardness removal is obtained per pound of salt used in the regeneration. This saving in operating cost is obviously made at the expense of initial cost because in such a case more zeolite would be required.

Furthermore, where regeneration is being made with impure brine, such as may be available in some localities in the form of sea water or brine wells, P is usually raised because with some Ca and Mg contamination in the initial brine, it is not possible to remove the Ca and Mg to the same extent from the zeolite. In such cases it is usually

best to operate at the resultant lower operating exchange value (OEV) because of the substantial saving in operating cost.

Similarly the point V may be shifted along the scale of exhaustion. As will be noted from the method for determining OEV, the standard

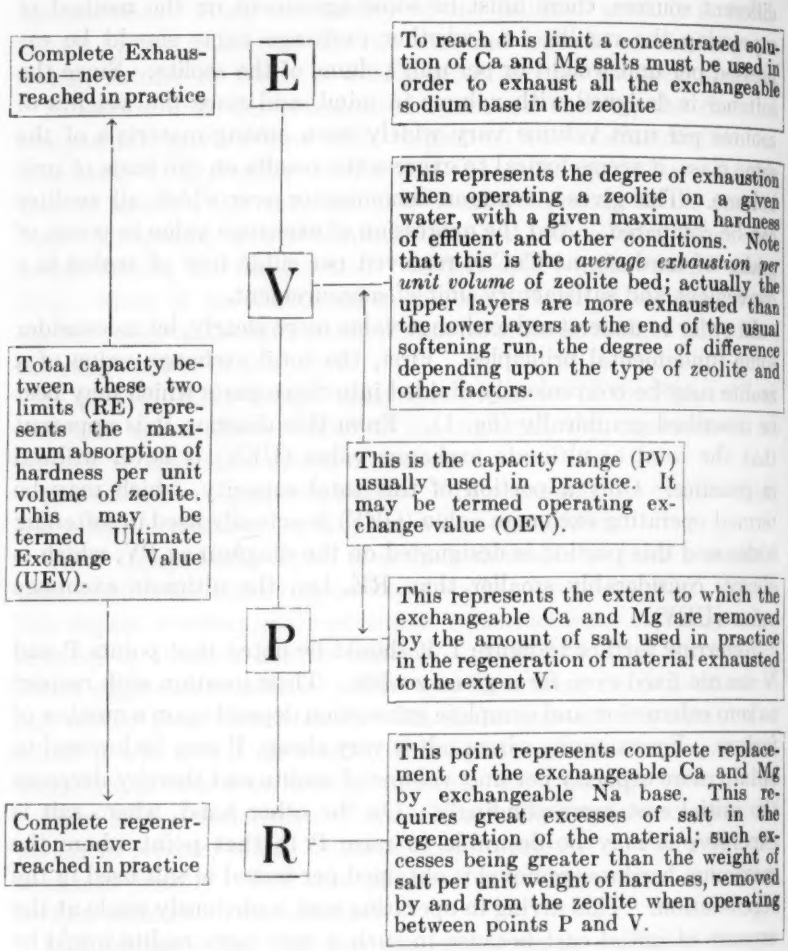

FIG. 1. DIAGRAM OF ZEOLITE EXCHANGE VALUES

test is made to determine the capacity when delivering absolutely soft water (i.e., a softening endpoint of about 0.5 grain). However, where absolutely soft water is not required (as in the case of municipal installations) it is possible to raise V (and perhaps P also) in order

to obtain a higher OEV as well as a lower salt consumption per 1000 grains (per kilograin) hardness removed.

Since the ultimate exchange value (UEV) is the maximum capacity obtainable, and since it does not necessarily bear any relation to OEV, which is the capacity utilized in normal operation, this UEV is usually of no particular importance. However, where a given grade of zeolite is being manufactured and it is desired to keep track of its quality, the method of employing UEV as a control may be useful.

Thus we see that operating exchange value (OEV) is not a specific figure even for a given zeolite unless the conditions under which the determination is made are clearly defined. Thus in considering this characteristic as one of the criteria in judging zeolite quality, one must consider OEV not as an isolated figure, but as a figure clearly and necessarily modified by definite stated conditions.

We have now considered the various factors which influence zeolite quality in a general or qualitative way. But quality in a zeolite as in everything else, is a relative term. What is best today may not be good enough tomorrow. And what may be high quality in one type of zeolite may be low in another. Therefore, the need of quantitative data as a measure of quality is apparent.[1]

The work described in this paper represents the joint efforts of the various members of the Permutit Research Staff. In this connection special appreciation is due Dr. Wm. M. Bruce, Dr. Paul C. Goetz, Dr. Ray Riley and Mr. Wm. Vaughan.

It is hoped that the members of the profession will view this work in the manner intended, namely as a starting point from which a much needed Test Code for zeolites may be gradually evolved as experience and study reveal further facts on these interesting and useful substances. If the present paper suggests various lines of approach and stimulates further development and a clearer understanding of zeolites, it will have served its purpose fully.

(*Presented before the Annual Convention, June 13, 1933.*)

[1] The detailed "Development of Test Methods" as well as the "Suggested Standard Methods" themselves have been omitted because of lack of space.—*Editor*.

DISCUSSION

A. S. BEHRMAN (Chemical Director, International Filter Company, Chicago, Ill): There is much in the paper by Mr. Tiger with which we agree most heartily. There are also, however, several statements and implications with which we must take sharp issue with Mr. Tiger. Unfortunately, time does not permit a full discussion of these points. In the few moments available, therefore, I shall confine myself to one or two portions of Mr. Tiger's paper which are most strikingly open to question.

Under the heading of "Wet Process Synthetic Zeolites," Mr. Tiger makes this statement: "Being of a very porous nature, these zeolites are also sensitive to clogging by suspended matter, organic matter or iron. Furthermore, if they are overrun, it requires a great number of regenerations to bring them back to their original condition."

The implication that has been stressed by so many ardent greensand enthusiasts that porosity in a zeolite is in itself a confession of inferiority, has amused us for a number of years, since our work in the development of both greensand and gel zeolites has brought us an appreciation of the facts of the situation which are not very generally understood.

In the first place, both greensand and the gel zeolites are very distinctly porous. A simple way of showing this is to dry thoroughly some of each material and then drop it into water. For quite some time afterward, you will notice streams of tiny bubbles arising, due to the replacement by the water of the air in the interior of both zeolites. There will be more bubbles per unit volume of the gel zeolite than of the greensand; but when it is remembered that it requires three or four volumes of greensand to give the same softening capacity as one volume of a modern gel zeolite, it becomes evident that there is more porosity per unit of softening capacity in the case of greensand than in the gel zeolite.

In the second place, the pores of greensand, as well as of the gel zeolites (as Mr. Tiger himself states in his paper) are "ultramicroscopic"—that is, they are too small to be seen even with a microscope.

As a matter of fact, particles of a typical gel zeolite look as transparent and homogeneous as glass even under the highest magnification we can employ.

In a paper presented before the American Chemical Society at Denver last summer, I pointed out that the work of Zsigmondy, Anderson, Butschli and Bachmann showed that the diameter of the pores of typical siliceous gels was around 5 to 6 millimicrons—in other words, only a few times larger than the largest true molecules. The smallest particles of suspended matter in water, on the other hand, such as clay, silt, etc. have diameters of the order of at least 100 to 150 millimicrons, and usually considerably higher.

In other words, there is just as much chance of clogging up a pore of either a gel zeolite or greensand with a particle of suspended matter in water as there is of stopping up a pinhole with a paving brick.

Every experienced manufacturer of zeolite softeners realizes that to use either greensand or a synthetic zeolite with a turbid water is contrary to sound practice, and is ultimately injurious; and Mr. Tiger's company has had ample opportunity for the demonstration of this fact.

The inaccuracy in Mr. Tiger's statement that synthetic zeolites, due to their porosity, are injured by turbidity, holds true also for the alleged effect of organic matter and iron in the case of the most efficient modern gel zeolites.

As far as the effects of overrunning are concerned, it must be emphasized that there are synthetic zeolites and synthetic zeolites. Some of them do appear to suffer under such circumstances, as Mr. Tiger has said; but some of them very definitely do not; and, if the synthetic zeolites of Mr. Tiger's acquaintance or manufacture suffer such injury on overrunning, the conclusion seems inescapable that he should try to make a new kind.

The second point in Mr. Tiger's paper which I wish to discuss very briefly is that of the durability of zeolites. It seems to me that there has been entirely too much loose and enthusiastic talk of the everlasting life of zeolites, whatever their nature. There is a great deal that we all have yet to learn about the durability of zeolites; but the one thing we do know is that they will not last forever. I am strongly in favor of Mr. Tiger's suggestion that an accelerated test be devised to give an idea of durability, provided that the test gives really fair and representative data. I feel sure, however, that the specific test he outlines—that is, treatment of the zeolite with an acetic acid-acetate buffer solution with a pH of 3.2—is not a fair test. It seems to me just as unreasonable to test a zeolite's resistance to pH 3.2, when waters actually to be softened have for the most part a pH

range of between 6.5 and 8.0, as it would be to test the cables of the Sky Ride at the Century of Progress by trying to see if they would support a locomotive or Pullman car.

Finally, in commenting on the somewhat individious comparison made by Mr. Tiger on the active life of greensand as compared with the synthetic zeolites, it should be borne in mind that the manufacturer of the best type of gel zeolites is making the same guarantees, and making them good, as is the manufacturer of the best type of greensand zeolites.

When all is said and done, we all know so little about the fundamental nature of zeolites and of the base exchange reactions that arbitrary statements should be made with extreme caution. Twenty or thirty years from now our successors in this field will probably be laughing heartily at our present mistakes and ignorance; and it is only by impartial analysis and observation, free from controversial self-delusion, that we will be able to achieve that more complete knowledge so earnestly to be desired.

ESTIMATING HARDNESS REMOVAL FROM SURFACE WATER SUPPLIES

By P. Charles Stein

(*Assistant Engineer, Whitman, Requardt and Smith, Consulting Engineers, Baltimore, Md.*)

Where accurate determination of relative softening costs is desired, as in the comparison of several proposed sources of water supply, it is necessary that the designer should know not only the nature and maximum concentrations of hardness, but also the relationships which exist between hardness variation and time. An adaptation of the stream-flow duration curve to give a hardness-duration curve as hereinafter illustrated is a useful instrument in the solution of such problems.

Surface supplies, being always the integration of variable quantities of surface run off, shallow subsoil drainage and deeply entrained ground water, all of which in their different courses acquire different mineral constituents, necessarily are subject to wide variations in their hardness concentrations. Many normally hard streams are sufficiently soft in high water periods to require no reduction treatment. On the other hand, it is not uncommon for the dry weather hardness of a stream to be five or six times that found in wet weather.

This article illustrates a typical problem involving the North and the South Forks of the Shenandoah River. These forks have their origins in Rockingham and Augusta Counties, Virginia and the drainage areas above the point of their confluence are 1036 square miles for the North Fork and 1653 square miles for the South Fork. The North Fork has, in general, a degree of hardness 26 percent greater than that of the South Fork.

The geology of the region is one where limestones and dolomites comprise the valley floor with frequent sandstone areas in the highlands. Many of the tributaries are fed by large springs flowing hundreds of gallons per minute.

Throughout the year of April, 1929 to March, 1930, the State Commission on Conservation and Development of Virginia in cooperation

with the United States Geological Survey analyzed samples from the North and South Forks of the Shenandoah, the results of which are published in Bulletin No. 3 of the Division of Water Resources and Power, of which Mr. J. J. Dirzulaitis is chief engineer. All the data

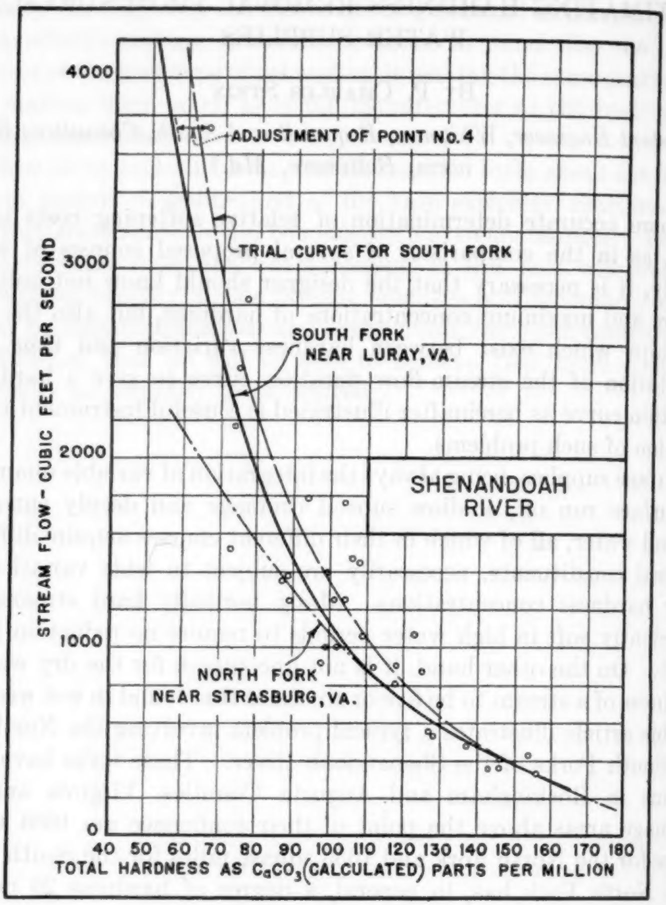

FIG. 1

used in this paper, as well as the general description of the watershed have their source in the aforementioned publication.

The procedure used by the Division consisted of the collection of daily 12-ounce samples and the admixture of these equal samples on the 10th, 20th and last days of each month, after which analyses were

made of the composite samples. In the aforementioned bulletin, total hardness is reported as $CaCO_3$ calculated from the concentrations of calcium and magnesium found in the composite samples. Average stream flow over the period of each composite sample is also reported.

Samples from the North Fork were collected at the stream gaging station near Strasburg, Va., above which point the drainage area is 772 square miles. On the South Fork samples were collected at the gaging station near Luray, Va., the drainage area being 1380 square

TABLE 1

DATE (MAY 1929)	STREAM FLOW, C.F.S.	HARDNESS FROM TRIAL CURVE, P.P.M.
1	1,900	85
2	1,800	87
3	11,400	30
4	6,810	48
5	3,940	61
6	3,200	67
7	2,520	75
8	2,140	81
9	1,910	85
10	1,690	90
Average..............	3,732	71

	p.p.m.
Hardness from trial curve at average flow of 3,732 c.f.s..........	62
Correction..	9
Original Plotting..	66
New Plotting = 66 − 9 = 57 p.p.m.	

miles. The concurrent discharge and hardness records at both stations make possible the determination of the effect of stream flow on hardness.

The first step in the problem of estimating hardness removal is the plotting of the calculated hardness given in the published data against the average stream flow during the period of each composite sample. The plotted points for the South Fork of the Shenandoah are shown on figure 1. A trial curve is then drawn approximating the trend of the points. As explained above, these points represent arithmetic averages of both discharge and hardness for the period of each

composite, the composite samples being made of daily samples of equal volume instead of daily samples with volumes adjusted for stream discharge. The result of the procedure employed is that the published data do not reflect the true effect of stream flow variations

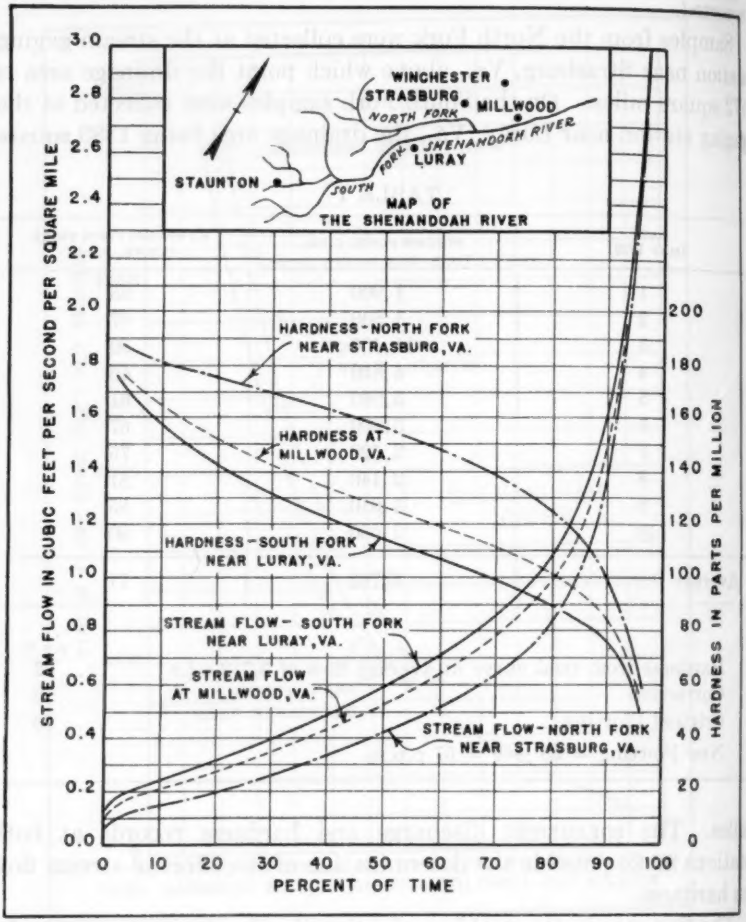

FIG. 2

on hardness, especially when it is realized that discharge variations over the periods of individual composites were as high as 2700 percent.

Since the stream flow-hardness curve, as drawn, is concave to the upper right hand corner of figure 1, any arithmetic average of several points necessarily will fall to the right and above the curve. Inas-

much as the trial curve gives the trend of ten day averages, a curve giving the trend of daily points must fall to the left of the trial curve. This adjustment can be approximated by determining the hardness from the trial curve for each daily discharge during any one sampling period. The arithmetic average of the hardness concentrations thus obtained can be compared with the hardness, also as read from the trial curve, corresponding to the average stream flow during the period of the composite. The difference in the two results is the error introduced by the curvature of the stream flow-hardness relationship and is also an approximation of the adjustment to be applied

TABLE 2

Calculation of hardness-duration curve

South Fork of Shenandoah River at Luray, Va., Drainage area 1380 square miles

DISCHARGE, C.F.S. (1)	C.F.S. PER SQUARE MILE (2)	HARDNESS, P.P.M. (3)	DAYS OF DEFICIENCY FLOW (4)	PERCENT TIME DEFIENCY FLOW (5)
200	0.145	180	10	0.55
300	0.217	158	113	6.19
400	0.290	147	309	19.62
500	0.362	137	474	25.96
600	0.435	129	628	34.39
800	0.580	115	874	47.86
1000	0.725	105	1115	61.06
1200	0.870	97	1324	72.51
1600	1.159	86	1498	82.04
2000	1.449	78	1581	86.58
2400	1.739	72	1670	91.46
3000	2.174	63	1709	93.59

to the plotting of the sampling result. As an illustration of the method involved, table 1 shows the adjustment calculations for the point labeled No. 4 on figure 2. The other points were adjusted by the same method and the final curve drawn on the basis of the adjusted points.

The discharge-hardness relation thus obtained can be applied to a stream-flow deficiency curve to derive a hardness-duration curve. For both the North and the South Forks of the Shenandoah River, stream discharge records are available in the Water Supply Papers of the United States Geological Survey for a portion of the year 1925 and for the succeeding water years to date. Table 2 shows the calcula-

tion for the stream discharge-deficiency and for the hardness-duration curves for the South Fork, at the gaging station near Luray, Va. In this table the first column gives the stream flows selected for which computations are made. The second column consists of the identical figures on a square mile of watershed basis. The third column shows the hardness of the stream coincident with the discharges in the first column as obtained from the curve on figure 1. The fourth column

TABLE 3

Calculation of hardness—duration curve for Shenandoah River at Millwood, Va., based on hardness—duration of north and south forks

PERCENT TIME	STREAM FL., CSM		TOTAL FLOW, C.F.S.		COMBINED FLOW		HARDNESS, P.P.M.		HARDNESS TIMES FLOW			HARDNESS AT MILLWOOD P.P.M.
	N. Fork	S. Fork	N. Fork 1036 sq. mi.	S. Fork 1653 sq. mi.	Total, c.f.s.	CSM 2689 sq. mi.	N. Fork	S. Fork	N. Fork	S. Fork	Shenandoah at Millwood	
0	0.053	0.098	55	162	217	0.081						
5	0.123	0.203	127	336	463	0.172	185.5	168.0	23,559	56,448	80,007	172.8
10	0.148	0.235	153	388	541	0.201	180.8	158.0	27,662	61,304	88,966	164.4
20	0.197	0.312	204	516	720	0.268	174.5	142.8	35,598	73,685	109,283	151.8
30	0.257	0.390	266	645	911	0.339	169.1	131.3	44,981	84,689	129,670	142.3
40	0.338	0.481	350	795	1,145	0.426	163.0	122.0	57,050	96,990	154,040	134.5
50	0.417	0.582	432	962	1,394	0.518	156.1	114.1	67,435	109,764	177,199	127.1
60	0.514	0.702	533	1,160	1,693	0.630	147.8	107.0	78,778	124,120	202,898	119.8
70	0.639	0.866	662	1,431	2,093	0.778	138.0	99.0	91,356	141,669	233,025	111.3
80	0.837	1.075	867	1,777	2,644	0.983	125.3	89.2	108,635	158,508	267,143	101.0
85	1.019	1.269	1,056	2,098	3,154	1.173	115.5	83.6	121,968	175,393	297,361	94.3
90	1.400	1.610	1,450	2,661	4,111	1.529	98.5	75.6	142,825	201,172	343,997	83.7
93	1.848	1.992	1,915	3,293	5,208	1.937	82.2	65.6	157,413	216,021	373,434	71.7

enumerates the number of days during which the stream flows in the first column were not exceeded in the periods included by water years 1926 to 1930 inclusive, a total of 1826 days. The last column represents the fourth column converted into percentage of the total time covered by the record, or the fourth column divided by 1826 days. This last column shows the percentage of time during which the stream flows in the second column were not exceeded and during which the hardness was not less than shown in the third column.

Figure 2 shows the plotting of the stream-flow-deficiency and the hardness-duration curves for the South Fork as derived in table 2 as well as similar curves for the North Fork near Strasburg, Va., and the Shenandoah River at Millwood, Va.

The curves for the Shenandoah at Millwood were derived by determining the weighted average hardness of the North and South Forks, the weighting being proportionate to the relative discharges of the forks, and an assumption being made that the deficiency-discharge relations of the two forks are concurrent. Although this is probably not true for any typical condition, it is believed that the curve thus derived gives a nearly true picture of the hardness-time duration relationship. The error introduced by this assumption is that occasioned when high water is caused by local conditions on the drainage area of one of the forks while low flows prevails on the other fork. Under this condition the hardness would be higher than that indicated on the Millwood curve on figure 2. This error, however, is of slight significance. Table 3 shows the calculation involved in deriving the Millwood curve.

Having obtained the hardness-duration curve, the average softening required to obtain a finished water of a desired hardness can be determined by planimetering the area bounded by the curve above that desired hardness. For example, to obtain a water not exceeding 80 parts per million hardness from the South Fork would involve softening for about 87 percent of the time. The area above the 80 parts per million ordinate bounded by the curve (before reduction for printing) is 6.03 square inches. Since on the original drawing, one part per million for 100 percent of the time is 0.1632 square inches, the average hardness removal would be 6.03 divided by 0.1632 or 36.9 parts per million. Reducing this figure to grains per gallon and multiplying by millions of gallons used per day and by the cost of such removal per grain per million gallons will give the daily cost of treatment. Required average hardness reduction for the North Fork near Strasburg and for the Shenandoah River at Millwood to produce finished water with a concentration not exceeding 80 parts per million is 68.5 and 46.1 respectively.

In order to justify the labor involved by the method outlined, the difference in results obtained by other procedures is pertinent. The arithmetic average hardness of the South Fork for the entire year covered by the analysis is 120 parts per million, and the weighted average based on average stream flows for the period of each com-

posite sample is 109 parts per million. These indicate average hardness reductions of 40 parts per million and 29 parts per million respectively in order to obtain a finished water of not exceeding 80 parts per million hardness, as compared with the figure 36.9 derived above. Of interest also is the average hardness of the South Fork of 114.6 derived by planimetering the entire area under the curve.

Similarly the average hardness and the weighted average hardness as determined from the analyses and the ten day average stream flows indicate softening to the extent of 56 and 42 parts per million for the North Fork compared with 68.5 derived by the duration method.

The method is particularly adaptable to several spot analyses made at such intervals as will reflect the effect on hardness of different rates of stream discharge. A half dozen samplings made at appropriate times together with concurrent discharge data will give an approximation at least of the average discharge-hardness relationship and establish the basis upon which the discharge-duration curve can be converted into a hardness-duration curve. This will enable a reliable solution of the problem to be made, whereas a few analyses are insufficient in themselves to delineate a true picture of the characteristic variations in hardness of the stream. In such cases where data are by no means comprehensive it is believed that the methods outlined should be of greatest value.

It must be recognized that stream flow conditions at times of increasing flow are entirely different than at times of diminishing flow and that, as a consequence, two equal flows, one on the increasing phase of the hydrograph and the other on the declining phase will be attended by different concentrations of hardness. In general, the hardness at times of increasing flow is greater than at times of diminishing flow. It is probable that seasonal and other factors also have considerable effect. Since this paper has endeavored to present a workable method, these effects have been ignored especially since the curves presented have been derived from actual data. However, in cases where spot analyses are the basic data, reliability will be considerably enhanced if samplings are made on both ascending and descending stages.

REPORT OF SECTIONAL COMMITTEE ON SPECIFICATIONS FOR CAST IRON PIPE, 1932

(Functioning under the American Standards Association)

TECHNICAL COMMITTEE 1. DIMENSIONS. W. C. HAWLEY, CHAIRMAN

Sub-Committee 1-A. Barrel of Pit Cast Pipe. T. H. Wiggin, Acting Chairman

A report was received from Professor M. L. Enger, on the strength tests made at the University of Illinois on 12-inch pit cast pipe. This was followed by a preliminary analysis of all the strength tests made on pit cast pipe for this committee.

A report was also received from Mr. W. J. Schlick on the trench load tests made at Iowa State College on 12-inch pit cast pipe. This was followed by a series of computations to determine proper thicknesses of 12-, 20-, 36- and 48-inch pit cast pipe, for different strength values of the iron, internal static and water ram pressures, combined with earth and truck loads and for different conditions of bedding and backfill. On a diagram these results were plotted so that a curve for a given size of pipe, strength of metal and external loads showed the thickness required for any internal pressure. Similar curves on the same diagram showed for comparison the thicknesses required by existing national and some local specifications.

Information was gathered and presented in a report by Mr. A. V. Ruggles on breaks on cast iron pipe and on installations where pipes in service have withstood without failure unusually heavy earth fills. The instances collected were mainly of larger pipe, where the effect of outside loads is more important and for each installation the factor of safety was computed against breaking under the combination of internal and external loads.

The indicated factors of safety thus obtained varied considerably, this being due in considerable part to the necessity of making assumptions without full knowledge of the size and shape of the trench, the method used at the time the pipe was laid of bedding and backfilling and the nature of the backfilling material. In quite a number of instances, however, where the pipes broke in service the

indicated factors of safety thus figured came out lower than would be considered satisfactory and indicated that the installations as made were not safe enough. The results of these computations will be used with the results from the trench load tests at Iowa State College to determine recommendations which will be issued by the Committee on how to choose the proper thickness of pipe to safely withstand the loads within and without which it must carry and how to determine, for the given soil conditions, the best size and shape of trench and methods of bedding and backfilling.

The same report contained details obtained of occurrence of water hammer in pipe lines in service and in addition included an inquiry on high pressure fire service distribution systems. The freedom from trouble on such systems indicates that the pipes used are strong enough, probably excessively strong.

Sub-Committee 1-B. Bell and Spigot Dimensions, Lugs and Harnesses. J. C. Proir, Chairman

Prof. Prior completed deflection tests of pipe as affecting bell strength and a test and preliminary report on a 36-inch steel Dresser clamp for preventing slip in a bell and spigot pipe joint.

A mathematical study and report were made by Mr. Joseph Goodman on "Consideration of Strength of Bells of Cast Iron Water Pipe to Resist External Forces."

Sub-Committee 1-C. Pipe Other than Pit Cast. F. H. Stephenson, Chairman

In March a program was adopted for tests of pipe other than pit cast. During the rest of the year this program received careful consideration and on account of business being so poor was curtailed as far as could be done and yield the results needed. Poor business prevented the producers from undertaking these tests as intended.

Sub-Committtee 1-D. Fittings. Charles Haydock, Chairman

Following the tests made for the Committee at Cornell University by Professor E. W. Schoder on friction loss through fittings, a study and report was made by Mr. A. T. Ricketts, with appendices by Mr. T. H. Wiggin, on the application of Prof. Schoder's results to the case of a typical water-distribution system. For this system, for both domestic and fire flows, determination of relative economy was made, including both cost of installation and capitalized cost of

pumping for two kinds of fittings; namely, long fittings as given in the present A. W. W. A. standards, which have 6-inch fillet radii at inside corners of tees and crosses and short fittings, with 2½-inch fillet radii, being considered by this committee.

TECHNICAL COMMITTEE 2. METALLURGY, PROCESSES AND TESTS. M. L. ENGER, CHAIRMAN

By the end of the year Technical Committee 2 and its four subcommittees had virtually finished their assignments save for recommendations for acceptance tests. Determination of acceptance tests to be substituted for or added to present ones depends upon the results of the strength tests which have been made for this Committee. In March, 1932, Prof. Enger submitted a preliminary analysis of all the tests on pit cast pipe made for the Committee, which, together with the other purposes which it will serve, will be a guide to choice of acceptance tests. Work on this problem continued throughout the year.

TECHNICAL COMMITTEE 3. CORROSION AND PROTECTIVE COATINGS. L. P. WOOD, CHAIRMAN

Sub-Committee 3-B. Organic Coatings. S. R. Church, Chairman

Throughout the year the best of the first set of pipes coated by the committee (in 1931) and all of the second set representing 6-inch pipes of four different methods of manufacture remained on the yard of the American Cast Iron Pipe Co. and were inspected from time to time.

The experiments begun in the previous year under Mr. J. T. MacKenzie's direction at the American Cast Iron Pipe Company plant to develop an accelerated test for pinholes were continued. Salt spray, one form of the ferroxyl test, and fuming with hydrochloric acid were tried without developing an entirely satisfactory method. Toward the end of the year the ferroxyl test as developed by Mr. Capron was tried with more promising results, and the producer members of the committee were requested to try the fuming test and report at the 7th Annual Meeting on their results with this and other pinhole tests. A digest was also made of the work on accelerated tests done during the past few years at the U. S. Bureau of Standards.

Sub-Committee 3-C. Inorganic Coatings. E. O. Sweet, Chairman

The test pipes and elbows containing the committee's experimental linings remained under flow test in the system of the Birmingham Water Company, throughout the year. It was not found practicable to make a second inspection of these pipes during this year.

At the Memphis convention of the American Water Works Association in May, 1932, a discussion of cement lined cast iron pipe and fittings was had during one of the Round Table Meetings, followed by a meeting of the Sub-Committee on May 4, 1932, to discuss criticisms and suggestions received relative to the Tentative Specification.

The Tentative Specification has continued in use by the cast-iron pipe manufacturers who are continuing their efforts to overcome shrinkage troubles on linings up to at least ¼-inch thick, to improve the lining of fittings, etc. At least one company, the U. S. Pipe and Foundry Co., has also been studying means of reducing solubility.

There were distributed to the members of the committee during the year a description of the cement lining practice of Warren Foundry and Pipe Corporation as described by Mr. Shellman B. Brown (under date of April 11, 1932, per vote of 6th annual meeting) and of the Naylor Pipe Company (under dates of December 17 and 30, 1932), and a description of the condition of some experimental neat cement and mortar linings of a 66-inch steel pipe after 17 years in Catskill water (under date of March 15, 1933)

RECENT STUDIES OF JOINT COMPOUNDS

By J. Hannan, Jr.
(*Consulting Engineer, Pleasantville, N. Y.*)

During the last few years the writer has been actively interested in the construction of water mains using compounds for jointing material. Observations have been made on the use of these compounds in the construction of some sixty miles of mains comprising distributing systems with pipe sizes ranging from 6 to 18 inches in diameter. The joint compounds used have consisted of only Leadite and Hydrotite, the two oldest compounds developed for joining cast iron mains. No significance should be attached to the fact that Leadite is mentioned first. This is simply a courtesy due old age.

There has been considerable discussion over a long period of years among water works men as to the relative advantages and disadvantages of joint compounds, as compared with lead. For purposes of discussion it would seem wise to review these advantages and disadvantages. It has been held that it is advantageous to use joint compounds because of —

The economy in construction
Their ability to withstand vibrations and temperature changes
Their durability, strength and tightness, and
The reduction of dangers due to blow outs during pouring

Certain disadvantages have also been advanced with reference to the use of this material, which are

The uncertainty of a satisfactory job if used by unskilled mechanics
The time required to carry out leakage tests
The difficulty of repair and salvage of joints, and
The prejudice of men skilled in the use of lead

This paper has been prepared with the idea in mind of setting forth certain studies of joint compounds, made expressly with the above points in mind, and to arrive at conclusions as to whether or not such statements are justified with regard to the particular work undertaken.

The ability of contractors to take laborers of average intelligence and make satisfactory joint men after simple instructions and a short period of competent supervision and practice, eliminates to a great extent the necessity for hiring highly paid men, supposedly skilled in the art of making joints. Despite the fact that there is uncertainty as to the result of the work when performed by unskilled labor, the amount of instruction and experience necessary to place a laborer in the skilled class, as far as the use of joint material is concerned, is so small that there is no question but that marked economy is achieved. Economy in the use of compounds seems to be generally admitted and the writer has encountered no serious arguments against this statement.

During the last three or four years the writer has let many contracts under specifications which allowed the use of either lead or joint compounds. The successful contractors, many of whom are highly skilled in the construction of water systems, have all elected to use compounds in preference to lead, the basic reason given being the economy involved. From observations of the work as progressed, economy is evident, due to the fact that no caulking and no large bell holes are required. The cost of heating the material and the cost of the joints themselves are less due to the lighter material. Information gathered from several water contractors indicate a cost of 16 cents per lineal foot per inch of diameter for completely installed mains with compounds, as against 18 cents for completely installed mains with lead or approximately a 10 percent saving.

VIBRATIONS AND SHOCKS

With reference to the ability of the joint compound to withstand vibrations and shocks, the writer has observed several significant cases, two of which are of particular interest.

It has been contended that the joint compound is unsatisfactory where the water line is laid under a road which may be subject to excessive vibration due to heavy traffic. In the Village of Chappaqua, Westchester County, New York, the writer has had occasion to observe the action of two lines laid in the same street. Ground conditions are extremely unsatisfactory, material is soft and the location is swampy.

About five years ago a 6-inch Class "C" water main was constructed through this street, approximately one thousand feet in length. This main was laid with lead joints carefully caulked and

carefully backfilled. About three years ago it was decided to repave this street and also lay an additional 6-inch main to reinforce the water system. This main was laid on the opposite side of the street in exactly similar soil conditions, but the joints were made with compound instead of lead. At the same time the joints on the older line were uncovered for an examination and these were found to be leaking badly and all had to be recaulked. A concrete pavement was then placed upon the street and the two lines have been in service continuously side by side since that time. Recent tests on these two lines have shown that the recaulked line is now leaking slightly and more than at the time it was recaulked and the line poured with joint compound has decreased slightly in leakage since its acceptance, at which time the leakage requirements for acceptance was one gallon per foot of joint per twenty-four hours. In this instance we have a very interesting case of two lines under identical conditions, subject to the same traffic, but joints of different material.

An interesting illustration of the ability of compound joints to withstand shock was observed by the writer during the construction of water mains at Millwood, Westchester County, New York. In this instance a hydrant had been installed on a 6-inch branch from the water main in very soft ground. About two months after the installation had been completed, satisfactory tests having been carried out on the line, the hydrant was hit by a large truck which pushed the hydrant over about 30° from vertical. An examination disclosed the fact that the hydrant had moved at the joint between the base of the hydrant and the 6-inch pipe connecting thereto, actually rotating about this point. There was approximately 150 pounds water pressure on the main and no leakage was observed. The contractor determined that he would attempt to restore the hydrant to its vertical position, simply by jacking it back in place. After considerable discussion and much uncertainty this was done by means of large jacks, the water pressure remaining on the hydrant all the time. The hydrant was successfully restored to a vertical position with no evidence of leakage whatsoever. It has now been functioning satisfactorily for a period of some two years and a recent examination of the joint shows it to be in satisfactory condition.

Conditions under which the work covered by this paper has been carried out have not made it possible to observe the action of joint compounds with reference to any extreme changes in temperature.

DURABILITY

This question of durability can only be determined by a long period of time. Records among many water works men who have used joint compounds over a long period of years do not seem to leave much question as to the durability of the material up to the present time.

With reference to the strength of joint compounds, the writer has had under supervision joints which have been subject to all pressures varying from 35 to 200 pounds per square inch working pressure. An interesting example of the ability of joint compounds to withstand pressures is furnished by an incident during which a 12-inch main was under a pressure test. The construction work was incomplete, branches having been left off for the hydrants and all hydrants, excepting one, having been set. The particular branch on which the hydrant was not set had a 6-inch gate valve on the end of a nipple, supposedly supported by suitable bracing. The contractor, through error, ran up a pressure of 400 pounds per square inch on this main when it was discovered that the gate valve referred to above had no bracing whatsoever. A careful examination of the joint showed that the valve had not moved in the slightest, although it had a load of 5.6 tons against it during the test, and the joints showed only slight evidence of sweating. This same joint has been in continuous service for some five years since that time with no evidence of leakage. A recently constructed 12-inch line is subject to surges in the amount of 500 pounds per square inch and has been in service for a year with no damage to the joints.

LEAKAGE

It is a recognized fact that lines laid with joint compounds must be filled with water in order to make them tight. An attempt has been made to determine the initial leakage immediately after water has been placed in the mains and many observations have been made on this subject. However, it is not possible to make any definite statement regarding the matter as the results have varied over such a wide range. The results of these tests have varied from 7.0 to 50.0 gallons per foot of joint, without any reasonable explanation for the variation. It appears at present that the only satisfactory method of determining whether a joint is satisfactory or not is by means of inspection while the line is under a pressure test, and then the best guide for acceptance or rejection of joints is the experience of the man making the inspec-

tion. One fact however is certain, and that is that the joints do take up after the line has been filled with water. After the joints have taken up, there is no question but that very satisfactory leakage tests can be obtained. A discussion of leakage brings us also to a consideration of the method of comparing leakage by the selection of a satisfactory unit.

TABLE 1

SIZE, INCHES	LENGTH OF PIPE, IN FEET		
	12	16	18
Leakage in gallons per inch mile based on leakage of 0.75 gallon per foot of joint			
6	102.2	76.6	68.1
8	100.4	75.3	67.0
10	98.5	73.9	65.7
12	97.2	72.9	64.8
14	96.6	72.4	64.4
16	96.1	72.1	63.9
18	95.6	71.7	63.7
20	95.3	71.5	63.5
Leakage in gallons per foot of joint based on leakage of 100 gallons per inch mile			
6	0.73	0.98	1.10
8	0.75	0.99	1.12
10	0.76	1.01	1.15
12	0.77	1.03	1.16
14	0.78	1.04	1.16
16	0.78	1.04	1.17
18	0.78	1.04	1.18
20	0.79	1.05	1.18

The inch mile basis has been used extensively throughout the water works field and appears to be satisfactory as long as we confine ourselves to standard 12 foot lengths of pipe. However, since the introduction of 16 and 18 foot lengths the inch mile basis does not correctly represent the joint leakage. The leakage in gallons per inch mile, based on a leakage of 0.75 gallons per foot of joint, and the leakage in gallons per foot of joint based on leakage of 100 gallons per inch mile are shown in table 1.

For example, with an allowable leakage of three-quarters of a

gallon per foot of joint, using 6-inch pipe, we would have an allowable leakage of 102 gallons per inch mile or with a 20-inch pipe of 12-foot lengths an allowable leakage of 95 gallons per inch mile. If similar pipe were installed in 16-foot lengths, the allowable leakage would be 77 and 71 gallons per inch mile, respectively. Similarly, should we elect to use 8-foot lengths the allowable leakage would be 68 and 64 gallons per inch mile, respectively.

TABLE 2

SIZE, INCHES	LENGTH, FEET	LEAKAGE IN GALLONS PER FOOT OF JOINT PER 24 HOURS
Tests run at pressure of 100 pounds per square inch		
6	1090	0.64
6	1000	0.61
6	2753	0.38
8	1298	
10	1003	0.73
16	1100	0.68 Av. 0.61
Tests run at pressures of 150 pounds per square inch		
6	1020	0.79
8	612	0.38
10	1066	0.84
12	1090	0.61
12	1300	0.61 Av. 0.65
Tests run at pressures of 180 pounds per square inch		
10	1300	0.50
10	1313	0.52
14	420	
12	1330	0.48 Av. 0.50

If we were installing 20-inch pipe in 18-foot lengths, based on a leakage specification of 100 gallons per inch mile, we could have approximately 1.18 gallons per foot of joint leakage and still remain within the specification. Here we have a considerable variation in leakages, as based on a unit of gallons per foot of joint, as against gallons per inch mile. There is no consideration in the above figures for joints due to fittings, and it would seem that a correct basis for determining joint leakage is gallons per foot of joint instead of the inch mile basis. All the leakage figures given in this paper are based on per foot of joint per 24 hours.

Several leakage tests which have been run on various mains constructed under the writer's supervision are given in table 2. The figures given have been selected from data compiled from many leakage tests and are presented as being representative of the results obtained. These lines were constructed under a specification which required a leakage of one gallon per foot of joint per 24 hours, which has been considered very satisfactory work with the use of joint compounds.

In all of the tests given in table 2 water had been in mains for a period in excess of three months.

The results in table 3 indicate that leakage is not permanently increased by increased pressure and that satisfactory leakage tests may be run on mains with compound joints between ten and thirty days

TABLE 3

SIZE, INCHES	LENGTH, FEET	TEST PRESSURE, POUNDS	WATER IN MAINS	LEAKAGE, GALLONS PER FOOT OF JOINT PER 24 HOURS
6	900	150	9 months	0.12
14	2000	180	9 months	0.11
16	3570	100	23 days	0.52
6	1012	100	15 days	0.72
6	2200	100	10 days	0.64
8	1694	180	10 days	0.95
6	1025	100	10 days	1.0

after filling with water. Experience indicates that the joints take up quicker if kept under pressure and several of the lines which have been unwatered for some time have shown no increase leakage when again put into service.

From the figures given in tables 2 and 3 leakage requirements on specifications for new work could be reduced to at least three-quarters of a gallon per foot of joint per 24 hours. Figures in table 1 will give the relative leakage in gallons per inch mile for this suggested specified leakage in gallons per foot of joint.

JOINTS WITH ENAMEL-LINED PIPE

Another decided advantage which has not been listed above, which is much more recent in development, is the success in using joint compounds with enamel lined pipe. A recent water system in Hawthorne, Westchester County, New York constructed under the

writer's supervision, is composed of water mains lined entirely with Bitumastic enamel. The pouring of these joints with compound has removed whatever faint possibility might exist of softening the lining and introducing any tendency to sag or drop. The range of temperature at which lead is poured is so great that it is extremely difficult to know exactly what temperature the lead may be. However, with joint compounds the proper time to pour is easily determined from the appearance of the compound when melted. The temperature ranged over which it may be poured is very small, in addition to being considerably below that of lead. In several instances, due to improper workmenship, the joint material has been allowed to run through the joint and spread over the bottom of the pipe. When these joints were removed and the pipe cleaned, there has been absolutely no damage to the lining or no difficulty in removing the joint material in one piece, as there appears to be no bond between the lining and the joint material. This interesting fact shows the necessity of proper cleaning of both the bell and spigot end of the water main if a satisfactory joint is to be made.

DISCUSSION OF DISADVANTAGES

With reference to the disadvantages which have been brought out concerning the use of compound for joint material, one of the chief arguments seems to be the uncertainty of producing a satisfactory joint unless the work is done by skilled men. The statement is correct, but as pointed out, the amount of training necessary to produce a man of sufficient skill to insure the fact that he can properly pour compound joints is so much less than that necessary to insure the fact that a man can satisfactorily caulk lead joints, that the writer does not believe this is a particularly serious argument. The same disadvantage exists in connection with lead joints, as anyone knows who has attempted to pour lead joints with unskilled labor.

Serious objections have been raised, particularly by contractors, who are anxious to progress as rapidly as possible, that the time necessary to lapse between filling the main and a satisfactory leakage test is a handicap as far as using joint compounds are concerned. This is certainly a fact if the Engineer insists that the joints must be left open until a satisfactory leakage test has been run. It has been the practice during the construction of a large portion of the water mains mentioned above, to simply insist upon a pressure test with a careful inspection of the joints before they are covered. Should the

pressure test prove satisfactory and the inspection of the joints by a man of reasonable experience show that they are reasonably tight, it has almost invariably been found that the leakage tests would be satisfactory within a period of from ten to thirty days after water has been placed in the mains. This seems to preclude the necessity of leaving joints open for any long period of time, and on work of any substantial size the fact that water should be left in the mains for even as long a period of thirty days before leakage tests are run should offer no serious handicap.

The question of salvage and repair of faulty joints is one that cannot be answered, except by the simple fact that unsatisfactory joints must be entirely removed and re-poured. The difficulty of removing joint compound is undoubtedly much greater than that of recaulking a lead joint. Compound manufacturers claim that a portion of the joint may be removed and re-poured and a satisfactory job will result. This generally has not been the writer's experience, except in cases where very low pressures have been used. Several attempts have been made to remove a portion of unsatisfactory joints and re-pour only that portion of them removed on lines carrying 150 pounds per square inch. In practically every case this has proven unsatisfactory. The joint would appear to be tight from the first inspection, but does not appear to be capable of withstanding the shocks to which a water system is ordinarily subjected.

The pouring of joints with compound on a suction line which is under a suction lift, has been attempted by the writer on only one occasion. On this occasion the pumps were subject to a suction lift of approximately 9 feet and after many attempts to prevent air from entering the suction line, the joint compound was removed from the joints and the line caulked with lead. Several attempts were made to have this line tighten by keeping it filled with water. This, however, did not prove successful. This line was laid in a location remote from all vibrations in a soil which was very satisfactory. The difficulty of accumulations of air in the pumps has been greatly corrected, due to the change in the joint material.

The writer has had very satisfactory experience with the use of joint compounds in practically all types of distributing systems, and is of the opinion that considerable savings can be affected, both in the cost of installation and in the amount of leakage to be expected by their use.

(Presented before the New York Section meeting, October 25, 1933.)

THE ST. LOUIS COUNTY WATER COMPANY PLANT

By W. V. Weir

(Superintendent, The St. Louis County Water Company, St. Louis, Mo.)

Fifty-seven years ago the largest city in St. Louis County was the City of St. Louis. In 1876, through an Act of the State Legislature, the City was allowed to separate herself from the rural area of the County. All the urban development was included within the new boundaries of the City along with what was erroneously considered enough undeveloped land to enable the City to spread to its supposed ultimate size.

Small towns in the territory not included in the new limits of St. Louis continued to develop along the steam railroads which carried commuters to work. Street car lines were later constructed connecting these communities with each other and with St. Louis. Development then took place along these car lines. Since the advent of the automobile and good roads the population has increased rapidly.

In 1899 sufficient development had taken place to warrant the consideration of a water supply system to serve portions of the County. Contracts to furnish water wholesale to two suburban towns were negotiated in 1901 and 1902 and a water supply system was begun by private enterprise.

In 1903 a steam pumping plant was in service at Howard Bend on the Missouri River. One 12-inch main to the distribution system carried all the water required.

In 1931 the steam plant, which had been enlarged several times, was superseded by an electric station. Important purification works were also built.

The original transmission main had been replaced after a few years and additional mains had been installed at various times until in 1930 three mains, 36, 24, and 20 inches in diameter carried water to the distribution system.

The system of the St. Louis County Water Company now serves 36,000 consumers in an area of 150 square miles. This area is two

and one-half times the area of the City of St. Louis. Six hundred and sixty miles of water mains are in service. Four standtowers and one elevated tank are required to maintain pressure in this large area in which the ground elevation varies three hundred feet.

LOW SERVICE PUMPS

The low service pumps are electric driven centrifugal pumps, three in number, which take suction from the river and discharge into the preliminary sedimentation basins. These pumps are mounted in houses on wheels which ride on rails at a ten percent grade. As the river stage rises or drops, the pump houses are moved along the track to maintain the correct suction lift. This construction affords a flexible, economical set-up where river stages have varied more than 33 feet.

Check valves on two of the pumps are on the discharge lines and one is on the suction. Pumps with checks on the discharge are primed by drawing vacuum, using ordinary air compressors as vacuum pumps.

These pump houses are heated electrically.

Winches are located at the upper end of each track by which means the houses are raised or lowered. Several discharge openings are spotted along each track for pump connections at different river stages.

Raw water is carried to the purification basins in 24, 20, and 16 inch cast iron lines and a 42 inch concrete pressure line.

TREATMENT PLANT

The preliminary sedimentation basins consist of six basins in parallel with a total capacity of thirty m.g.d. at 3 hours detention. These basins are each 37 feet wide and 150 feet long, with a water depth of 15 feet. Basins of this shape operate with no short circuiting and give high deposition efficiency.

These basins are mechanically cleaned by means of scraping flights which move longitudinally, scraping and moving the sludge to hoppers at the inlet end of each basin. Sludge is removed by gravity from these hoppers and is discharged upon land to be filled or is sent to the river.

This type of sludge removal equipment has not been used as extensively in water purification as it has been used in sewage treatment. The waterworks profession have generally used round or

square sedimentation tanks with rotating sludge removal machinery. Due, however, to the fact that basin shapes are not limited to either round or square, this type of sludge removal mechanism will be used more extensively in the future.

Our installation could have been made into one large basin by eliminating the division walls. These dividing walls may be considered as the means to straighten and control the water flow, and they also allow inspection and repairs to be made reducing capacity only one-sixth.

In this locality ice often causes trouble where basins are mechanically cleaned. In this installation of longitudinal flights, the return flights are at least four feet below the water surface, and are entirely unaffected by ice. The only moving part which touches the water surface is the main drive chain from the head shaft above the basins to the flight shaft sprocket. This chain is heavy and travels slowly. The sprocket on the head shaft is housed and equipped with electric strip heaters to prevent or melt ice formation. These heaters have been used only in sub-zero weather in the past two years.

The scraper speed is variable from 6 to 18 inches per minute by means of variable speed reducers between the motors and the head shafts.

It has not been found necessary to operate the removal machinery continuously. One pass of the flights in eight hours will sometimes handle all sludge deposited. No trouble is encountered in operating the machinery after heavy sludge depths have been allowed to accumulate in the basins. Such sludge depths are not normal operating practice. Four feet of heavy sludge have been accumulated at times of experimental runs. No overloading of motors or equipment has occurred in starting the sludge removal mechanism.

Milk of lime is usually applied to the influent raw water. Softening occurs and coagulation takes place.

Sludge concentrations obtained this year have averaged 25.2 percent dry solids with a maximum of 47.4 percent. The preliminary basins account for a removal of 93.8 percent of total suspended solids in the last six months.

Effluent water from these basins is measured by a recording weir meter, the recording device being installed in the chemical feed room so that dosages may be varied with water flow.

Coagulant, either iron sulfate or alum, is applied above the weir to take advantage of the violent mixing below the weir. The dosed

water then passes through an around-the-end baffled mixing basin having a detention period of 20 minutes. Passes in this chamber are wider toward the end of the water travel.

Some trouble is encountered in the transportation of this conditioned water through a 60-inch conduit to three sets of coagulation basins approximately 150 feet away. Floc is broken up to some extent, but sufficient gentle agitation is provided in the inlets to the coagulation basins to recondition the water very satisfactorily.

From the coagulation basins the water passes through another around-the-end baffled mixing chamber into a 40 million gallon clear water reservoir. Recarbonation takes place in the first passes of this chamber. A small dose of alum is sometimes applied to water in this chamber if the effluent from the coagulation basins has a noticeable turbidity.

Sterilization is accomplished by application of ammonia and chlorine to the coagulated water with a second application of ammonia and chlorine to the water as it enters the high service pumps. A third application of chlorine is sometimes applied to water entering booster pumps between the transmission mains and the distribution system if the chlorine residual at that point is not high enough. Ammonia is obtained at one point by dosing ammonium sulfate solution. The other application is made by feeding aqua ammonia. Chlorine is obtained in ton containers.

Routine chemical and bacteriological tests are made daily so that close and accurate control may be had over the water during its passage through the various stages of the purification process.

The chemical house is a temporary structure housing chemical storage and the various chemical feed machines. A new chemical plant is scheduled for construction in the very near future.

HIGH SERVICE PUMPS

The high service pumping station has several features of probable interest. This station is electric in all respects, even including heating.

Current is purchased at 33,000 volts and is stepped down to an operating voltage of 4,000 volts.

Due to the necessity of elevating the electric substation to a point above the possible high water, which is several feet above the general ground level, the substation was placed on the plant roof. This construction has been very satisfactory as to operation and also in regard to appearance.

Two high service pumping units are installed. The station has been constructed for future extension to the South and to enable the ultimate installation of three or more additional units.

Three single stage 18-inch volute pumps, driven by two motors, make up each pumping unit. The first stage pump in each unit is driven by a 550 h.p. variable speed induction motor. The second and third stage pumps are driven by 1100 h.p synchronous motors installed between the pumps.

The motors operate at 4,000 volts. Across-the-line starters are used on these motors, no trouble of any kind being experienced.

The check valves on the pump discharge lines are 24″ x 16″ Larner-Johnson valves. Pump discharge is measured by utilizing the head differential across these valves. Check tests with pitot tubes and gaugings on the reservoir have indicated the accuracy of these meters through the entire discharge range.

The efficiency of these variable speed units has proven to be very satisfactory. The overall efficiency of water output to electric input has been 74 percent for the first nine months of 1933. This is the combination motor efficiency and pump efficiency through all the high service pumping operations.

Two separate high tension power lines run from St. Louis to the Plant. Separate sources of power feed each of the lines, which are also so inter-connected that any power plant can feed over either line.

Spare equipment is installed in the Plant so that any breakdown of electrical units can be quickly repaired. Of four main transformers on the roof one is a spare, and transfer track is provided so that it may be quickly placed in its proper position, if necessary.

We have kept the old steam plant with its boilers and reciprocating machinery in full standby condition, all units ready to go at any moment. This plant has been in operation twice due to emergencies. Failure of the new plant was caused each time by pipe line trouble and not by electric trouble or power failure.

The new sedimentation basins and the high service pumping station were designed and constructed by the United Engineers and Constructors, Inc. At the time these units were built the old steam plant was overhauled by them to be in complete ready-to-go condition.

(*Presented before the Missouri Valley Section meeting, October 27, 1933.*)

Myron Borland Reynolds

1880-1934

Mr. Reynolds, late city engineer of Chicago, was born December 12, 1880, at Pana, Illinois. He was educated in the public grade and high schools at Pana. After two years at the Rose Polytechnic Institute at Terre Haute, Indiana, he transferred to Armour Institute and was graduated with honor in 1906, receiving a B.S. degree in Civil Engineering. He was elected to the first chapter of Tau Beta Pi at the Institute. Two years later he received his C.E. degree at Armour.

Mr. Reynolds first entered the employ of the City of Chicago as a Grade 7 Civil Engineer through Civil Service appointment on June 21, 1907. Except for two leaves of absence, when he engaged in private contracting work and fifteen months of military service during the World War, Mr. Reynolds continued in the engineering service of the City of Chicago from 1907 until his death on January 27, 1934.

From 1907 until 1909 he was employed on outside construction work on water tunnels. From November 1911 to October 1913 he served as Assistant Engineer both on design and construction with the Board of Local Improvements in connection with the Michigan Avenue Improvement project. He was promoted to the position of Engineer of Water Works Design in the Bureau of Engineering on October 2, 1913. In this capacity he was in responsible charge of the design features of some very important improvements in Chicago's Water Works System, among them the Wilson Avenue Tunnel and Crib costing $3,900,000; the Mayfair pumping station, the new Lake View pumping station and the Western Avenue Tunnel. He directed very important engineering studies on the hydraulics of flow of water through large tunnels. It was during Mr. Reynolds' service as engineer of water works design that plans were made to modernize and increase the capacity of many of the older pumping stations by installing motor and turbine driven centrifugal pumps in place of the older triple expansion pumping engines.

In September, 1917, he obtained a leave of absence to enter the Second Engineer Officers Training Camp at Ft. Leavenworth, Kansas, receiving a Captains commission. He was detailed as an instructor at the 3rd and 4th Engineer Officers Training Camps at Camp Lee, Petersburg, Virginia from November 1917 to July 1918. He was promoted to Major in the Engineer Corps in the fall of 1918, and also served as instructor at Camp Humphries, Virginia. He was honorably discharged from military service at Camp Travis, Texas in January 1919, and returned to the engineering service of the City of Chicago.

On October 4, 1923 Major Reynolds was temporarily appointed assistant city engineer under the late John Ericson, long city engineer for Chicago. This was a well merited advancement for he had organized a very efficient designing division. He was promoted to full rank as assistant city engineer on January 21, 1925. After the death of Mr. Ericson in April, 1927 Major Reynolds served as acting city engineer until August of that year. On October 9, 1931 he was promoted to the position of City Engineer, the highest grade in the engineering service of the City of Chicago.

Mr. Reynolds as a principal assistant to Mr. Ericson was responsible for many of the recommendations during the last twenty years which brought about the modernizing of Chicago's water works system into its present efficient units. He was a strong advocate of metering and filtration and did much during his term as city engineer to advance these important projects.

Mr. Reynolds was a strong advocate of pure water for Chicago. He supported chlorination as a public health measure when it was very unpopular with the public. With Mr. Ericson he urged the construction of Chicago's experimental filtration plant, the construction of which was started before Mr. Ericson died. During the last two and a half years under Mr. Reynolds, filtration for Chicago was advanced to the actual submission by the city of Chicago last December of an application to the Federal Emergency Administration of Public Works for a loan and grant totaling $22,400,000 to build a filtration plant for the south water district. This project included complete meterization of all consumers in the district to be served. The approval of this great project by the City Council, including the unpopular meter measures, was ample evidence of the confidence the city officials had in Mr. Reynolds. It was also another demonstration of his courage in carrying forward to victory, in spite of its

political disfavor, a measure he knew to be right and for the best interest of the city.

Mr. Reynolds was a member of the American Society of Mechanical Engineers and the American Water Works Association and contributing freely of his work and experience to the engineering literature.

It was largely through Mr. Reynolds' interest and energy that the 1927 and 1933 conventions of the American Water Works Association were such outstanding successes. As Acting City Engineer in 1927 he took over the responsibilities of City Engineer John Ericson as Chairman of the Chicago Convention Committee after the latter's death in April of that year. Mr. Reynolds was Chairman of the 1933 Chicago Convention Committee.

Through his untiring efforts arrangements were consumated for the dedication of three bronze memorial plaques to former city engineers of Chicago on the opening day of the A. W. W. A. Convention June 12, 1933. At that time, with fitting ceremonies, these three plaques in memory of Ellis S. Chesbrough, DeWitt C. Cregier and John Ericson were placed on the old Water Tower, Chicago's famous landmark.

Because of the holding of the conventions of numerous professional engineering societies in Chicago during the period of the World's Fair in 1933, Mr. Reynolds as City Engineer, caused to be prepared by the Bureau of Engineering a well illustrated history of Chicago's Water Works. This brochure was distributed to delegates and to hundreds of public and professional libraries in this and other countries.

A few days before his death on January 27, 1934, Mr. Reynolds was awarded the 1933 medal of the Illinois Society of Engineers as senior author of a paper "Chicago's Water Pollution Problem—Past, Present and Future," selected by the award committee as the best paper published by the society for that year.

Major Reynolds was Commander of the Castle Post 151 of the American Legion, whose membership consists of professional engineers. He was a true leader of men, modest, capable and kind. His influence on men and matters of engineering policies will be carried for many years in the Bureau he so ably directed.

Mr. Reynolds is survived by his mother, Mrs. Susan Geist Reynolds of Pana, his wife, Irene Periolat Reynolds, and his son, Myron B. Reynolds, Jr., age 8 years.

Arthur E. Gorman.

ABSTRACTS OF WATER WORKS LITERATURE

Frank Hannan

Key: American Journal of Public Health, 12: 1, 16, January, 1922. The figure 12 refers to the volume, 1 to the number of the issue, and 16 to the page of the Journal.

Proposed Method of Tunneling. Eng. News-Rec., 107: 146, July 23, 1931. Brief outline of novel method for rapid and economical tunnel work in rock, or other material that does not require timbering. Outstanding features of method are: (1) inclined overhanging tunnel face; and (2) use of traps, of special skips, into which blasted material falls. Chief advantages claimed are: (1) ability to pull, or advance, face 12 feet horizontally by means of drill holes only 6 or 7 feet deep; (2) use of greater number of drills at one time, due to increased surface of tunnel face; (3) not more than 2 sizes of drill steel required, due to relatively short holes; (4) reduced time of setting up drills, due to use of stopper operated from platform for most of holes; (5) reduction in quantity of powder, due to more advantageous location of charge; (6) less overbreak; (7) less smoke and gas; (8) greatly reduced mucking cost. Method was worked out by H. H. Fischer. Although method has been patented, it is understood that permission to use it will be granted without charge to reliable contractors.—*R. E. Thompson.*

World's Largest Double-Track Tunnel Pierces Italian Apennines. Eng. News-Rec., 107: 65–6, July 9, 1931. Data are given regarding 11.5-mile railway tunnel on Florence-Bologna line of Italian state railways through Apennine Mountains. Tunnel has horse-shoe section about 20 feet wide at springing lines and a 4000-foot section near center is widened to about 56 feet. Lining is brick and stone masonry, latter being employed where pressure was greatest. Total excavation was about 1,950,000 cubic yards and volume of lining 520,000 cubic yards. Tunnel was driven from portals and from center shafts. Difficulties were experienced with large inflows of gas and water, and the ground developed enormous pressures. The gas, which was of methane character, had to be burned off at one place every two hours by electric ignition. Project was begun in 1923 and completed in 1928. Cost was \$24,735,000.—*R. E. Thompson.*

An Early Yuba River Flood. W. W. Waggoner. Eng. News-Rec., 107: 305, August 20, 1931. Yuba River has mountain watershed of about 1200 square miles, embracing region of heaviest precipitation in the Sierra Nevadas. Average annual rainfall varies from 40 inches in foothills to 75 inches at elevation of maximum precipitation. During December, 1861, and the following January, about 75 inches of rain fell on lower Yuba drainage area, and during

the same period about 42 inches of rain and 50 feet of snow fell at summit of the Sierra. Total precipitation for the season in the mining territory reached 109 inches; rains of between 5 and 6 inches per day were not uncommon. From foothills of Mount Shasta south to base of Tejon Pass, great central valley of California was vast lake, not unlike Lake Michigan in shape and size. Investigations in 1894 revealed deposits, produced by floods and hydraulic mining operations, up to 80 feet in depth. Solution of general flood problem, author believes, is construction of reservoirs to provide for control of flows which cannot readily be handled by downstream channels. Gaging records of past two decades are not sufficient indication of floods which should be anticipated. —*R. E. Thompson.*

For 1931: Extensive List of Water and Sewage Works under Way or Planned. A. E. BERRY. Cont. Rec. and Eng. Rev., 45: 288–90, 1931. Brief outline of water and sewage disposal works in progress, or proposed, on Ontario.—*R. E. Thompson (Courtesy Chem. Abst.).*

First Floodflow Experiments at Vicksburg Hydraulic Laboratory. Eng. News-Rec., 106: 970, June 11, 1931. Brief data given regarding first two studies carried out in Vicksburg laboratory: (1) erosion of railway embankments by floodwater; (2) limit of backwater effect from maximum all-time stage of Mississippi River against maximum floodflow of Illinois River and against flows of 1904 and 1926.—*R. E. Thompson.*

Concrete Lining for Irrigation Canal in Texas. A. J. MOORE. Eng. News-Rec., 107: 60–1, July 9, 1931. At least 250 miles of canals and lateral ditches have been lined with concrete within past 3 years in Hidalgo and Cameron counties, Texas, while about 800 miles are planned for these and other counties of Lower Rio Grande valley. This practice is proving to be solution of valley's greatest problem, seepage prevention, and has created new standards of efficiency and economy in water distribution. Brief details of lining methods included.—*R. E. Thompson.*

Irrigation in Java. HAROLD E. BABBITT. Eng. News-Rec., 107: 13–16, July 2, 1931. Outline of irrigation practice in Java. In March, 1930, there were 42 irrigation projects under construction, involving expenditure in excess of 40,000,000.—*R. E. Thompson.*

Water Line for Sharonville-Springdale, Ohio. Eng. News-Rec., 106: 949, June 4, 1931. Unit prices given from bids on cast iron pipe lines for villages of Sharonville and Springdale and for distribution system for former.—*R. E. Thompson.*

New Permeability Specimen for Comparison Tests. CLOYD M. CHAPMAN. Eng. News-Rec., 107: 9, 1931. Brief description of bomb type permeability specimen devised by author, which enables dependable comparisons to be obtained. Specimen is molded around long-necked toy balloon filled with exactly 140 cc. of fine, dry, free-flowing sand, which gives a spherical cavity 2½

inches in diameter, having surface area of 20 square inches. Outside diameter of specimen is 6 inches and height, 6 inches. When concrete is hard, sand is poured out, bag withdrawn, and cement skin removed from the surface by washing with dilute HCl and rinsing with water.—*R. E. Thompson (Courtesy Chem. Abst.).*

Lining of New Albany Reservoir to be Reinforced. Eng. News-Rec., **107**: 312, August 20, 1931. Use of new upland water supply system of Albany, N. Y., has been postponed due to unexpectedly large leakage from Loudenville distribution reservoir, which consists of two basins holding 75 and 25 million gallons, respectively. The bottom construction, consisting of two 4-inch concrete slabs separated by several layers of waterproofing membrane, has proved satisfactory, but the side slopes, built of single 8-inch thicknesses of concrete, have developed considerable leakage through the joints of the 16 x 40-foot panels. No percolation through slabs has been detected, nor is any settlement evident. Joint construction includes 10-inch continuous crimped copper strip embedded in the concrete and ½ x 8-inch premolded bituminous filler. Although only a small proportion of 41,000 linear feet of joints has proved defective, it has been decided to cover entire surface with 1½ inches of gunite.—*R. E. Thompson.*

Reinforced Concrete Cover for Two Reservoirs. Eng.-News-Rec., **107**: 119, July 16, 1931. Two adjoining basins, each 415¾ x 502½ feet in plan and 36.42 feet high, of Compton Hill Reservoir, St. Louis, are being provided with concrete cover at cost of $222,440. Complete unit prices are given.—*R. E. Thompson.*

Water Softening Plant Employs Pumped Recirculation for Chemical Mixing. Harry N. Jenks. Eng. News-Rec., **106**: 1061–3, 1931. Recently completed water purification plant at Fort Dodge, Ia., is described. Raw water, derived from wells, has iron content of from 0.5 to 3.0 p. p. m., total hardness of 550 p. p. m., and contains H_2S and CO_2. Plant consists of a cascade aërator, from which water flows in thin layer over channel strewn with pebbles and small rocks which effectively riffle surface, agitation tank in which water is recirculated over riffled concrete slab, two series-connected mixing tanks, in which spiral flow is induced by recirculated water discharged tangentially near water surface, two sedimentation tanks equipped with sludge scrapers, with recarbonating chamber located in connecting channel, four rapid sand filters, and 2-million-gallon filtered water reservoir. Softening is not at present practised. Addition of coagulant is not necessary, natural iron floc being sufficient for clarification. All iron and H_2S are removed and CO_2 content is reduced by 25 p. p. m. Plant was designed on basis of results obtained in 2-gallon-per-minute experimental plant, cost of which was $600. Contract price for plant was $143,850. On basis of 4 million gallons per day capacity, cost of plant for iron removal alone worked out at approximately $25,000 per m. g. d., and for both iron removal and softening, at $35,000 per m. g. d.—*R. E. Thompson (Courtesy Chem. Abst.).*

Long-Range Planning Essential in Developing Municipal Utilities. JOHN H. GREGORY. Eng. News-Rec., **106**: 845–8, May 21, 1931. Data are given regarding Baltimore, Boston, Detroit, New York, and San Francisco water supply projects, showing that from 10 to 15 years, or longer, is often required to bring to completion such large undertakings. Data were compiled in connection with Baltimore water supply extension, for which enabling act for expenditure of $27,500,000 has been submitted to Maryland legislature—*R. E. Thompson.*

Hydraulic Lime in Concrete. G. W. HUTCHINSON. Eng. News-Rec., **106**: 974–6, 1931. Results of laboratory study of value of hydraulic lime in combination with portland cement in making of concrete are shown graphically and discussed. Additions of hydraulic lime up to 25 percent increase compressive strength of concrete; above this point, strength falls off, due to increase in volume of concrete and consequent reduction in portland cement content. Addition of 50 percent hydraulic lime to lean mixture provides same strength as 50 percent increase in portland cement, cementitious powder content per cubic yard of concrete being 1½ cubic feet more in former than in latter case. With given portland cement content and with strength the sole consideration, paste content would be found too low in most cases to meet requirements for workability, watertightness, uniformity, etc. By adding hydraulic lime, paste content irrespective of strength, but consistent with other requirements of concrete may be maintained. Admixtures of hydraulic lime also reduce undesirable effects of increased water:cement ratio.—*R. E. Thompson (Courtesy Chem. Abst.).*

Earth Slips and Subsidences from Underground Erosion. KARL TERZAGHI. Eng. News-Rec., **107**: 90–2, July 16, 1931. If the ground below a dam, or natural barrier, to impounded water contains layers or zones more permeable than surrounding soils, they become the seats of intense underground currents. Physical action is as follows: starting at point where zone of weakness opens into free drainage, erosion works backward into groundwater carrier, scours cavity in permeable stratum and ultimately brings about collapse of roof of cave. Subsidence and slip due to underground erosion in high bank of Mississippi River at Memphis, Tennessee, and failure of Corpus Christi dam are described and discussed.—*R. E. Thompson.*

Corpus Christi Dam Failure. J. B. LIPPINCOTT. Eng. News-Rec., **107**: 150, July 23, 1931. Brief discussion based on observations made by writer some months after failure occurred.—*R. E. Thompson.*

Treating Taste and Odor in Public Water Supplies. NORMAN J. HOWARD. Cont. Rec. and Eng. Rev., **45**: 501–3, 506, 1931. General discussion of various tastes and odors occurring in water supplies, particularly those following chlorination, their causes and methods of prevention, or removal, including superchlorination preammoniation and activated carbon filtration.—*R. E. Thompson (Courtesy Chem. Abst.).*

Inflows Block Construction of Aqueduct Tunnel for Athens, Greece. R. K. Keays. Eng. News-Rec., 106: 978–81, June 11, 1931. Detailed description of difficulties experienced during construction for new water supply of Athens of Boyati tunnel, 8.37 miles long and of horse-shoe section, 7.54 x 7.87 feet inside, with lining of pre-cast concrete segments, set 6 to a ring. Heavy flows of water were encountered which filled heading and completely blocked tunnel with fine black silt. Flowing chlorite schist, which resisted all attempts to control it, finally led to diversion of north portal heading. In note by Editor, new water supply system of Athens, which is being developed to supplement that constructed in time of Emperor Hadrian and continuously used since second century, is outlined as follows: (1) Marathon dam, marble-faced monumental masonry structure of gravity section, 177 feet high; (2) reservoir, impounding 41,000,000 cubic meters; (3) Boyati tunnel; (4) control works at Boyati south portal, delivering to Halidonou tunnel aqueduct, which, including short Kokinara aqueduct bridge, is 1.55 miles long; (5) cast iron inverted siphon, 36 inches in diameter and 3½ miles long; and (6) filter plant and service reservoir for Athens and Piræus.—*R. E. Thompson.*

Improvements in Rapid Filter Design. Robert Spurr Weston. Cont. Rec. and Eng. Rev., 45: 289, 1931. Brief discussion of present practice in rapid sand filtration. Trend is, to depend more and more on gravel layers for distribution of wash water and to employ coarser sand and higher rates of filtration. Sand of 0.55 mm. effective size is being successfully employed at several plants.—*R. E. Thompson (Courtesy Chem. Abst.).*

Three Types of Design Used in California Flood-Control Dam. Ralph G. Wadsworth. Eng. News-Rec., 107: 46–9, July 9, 1931. Illustrated description of Hogan dam, recently built by Stockton, California, on Claveras River, for flood control. Structure, which is 137 feet high above lowest foundation, embraces 3 separate types of construction: variable radius arch with massive gravity abutments, rolled earth-fill, and standard gravity sections, both straight and curved. Reservoir capacity will be 76,000 acre-feet.—*R. E. Thompson.*

Present Tendencies in Coagulation. Robert Spurr Weston. Cont. Rec. and Eng. Rev., 45: 285, 1931. Brief outline of trend in practice in regard to coagulation of water. Requisites for good coagulation include: proper reaction (pH), complete mixing, flocculation, and subsidence. Slow stirring in tanks by hydraulic, or mechanical, means after mixing is essential.—*R. E. Thompson (Courtesy Chem. Abst.).*

A New Water Supply in Ten Months. Malcolm Pirnie and Charles F. Ruff. Eng. News-Rec., 106: 842–4, May 21, 1931. New water supply system for St. Petersburg, Florida, was constructed by Pinellas Water Co., in less than one year under contract by terms of which company has exclusive right for 30 years to supply all water wholesale to city for distribution through existing mains. Contract specified water of less than 160 p. p. m. hardness and of less than 25 p. p. m. salt content, and called for immediate capacity of

10 and ultimate capacity of 20 m. g. d. Project consists of 12 wells across old Tampa Bay, well pumps, aërator, 26-mile 36-inch pre-cast concrete pipe line to city, and storage reservoir and pumping station near city to raise water to distribution pressure. The wells, spaced 1000 feet apart, have average depth of 300 feet and yield from 0.75 to 2.0 m. g. d. each. Peak capacity of system is 16 million gallons per day. Each well is equipped with deep-well pump placed from 40 to 50 feet below ground surface. Drawdowns vary from 6 to 25 feet. Water is delivered to aërator located on roof of 300,000-gallon steel tank which feeds reservoir in city by gravity through pipe line. Chlorine is applied as water enters conduit. Pipe was made up of 1½-inch concrete lining, cylinder of 14-gage steel that had been welded and air-tested for leaks, and 2½-inch concrete covering in which was placed steel reinforcing. Leakage totaled 8,000 gallons per day, or 10 gallons per inch diameter per mile, under operating pressure. Average value of *C* (Williams-Hazen formula) was 150.4. City reservoir is steel tank, 117 feet in diameter and 42 feet high, having capacity of 3.5 million gallons. Its function is to permit operation of well field at uniform rate. Pumping equipment consists of three motor-driven centrifugal units of 3, 6, and 13 million gallons capacity, respectively. Tests showed overall efficiencies of 77, 76.2, and 83.5 per cent. Former supply, derived from wells in city, had gradually become hard and very saline.—*R. E. Thompson.*

Vehicle Tunnel to be Built under the Scheldt at Antwerp. Eng. News-Rec., 106: 1003–4, June 18, 1931. Brief details given of proposed tunnel, which is to be of shield-driven type, with cast iron lining. Total length between portals will be 5800 feet, and between street grades, 6922 feet. Major portion of tunnel will be in clay and maximum depth below mean water level will be about 90 feet. Smaller tunnel will be driven about ½ mile upstream for pedestrian traffic.—*R. E. Thompson.*

Coldbrook-Swift Tunnel Near Boston. Eng. News-Rec., 106: 1033, June 18, 1931. Unit prices given from bids received by Massachusetts Metropolitan District Water Supply Commission on contract for 9.5-mile extension of Wachusett-Coldbrook tunnel which was let for $4,978,032. Tunnel excavation amounts to $3,388,000, or 68 per cent of contract. Contract unit price was $8.80 per cubic yard of excavation.—*R. E. Thompson.*

Madden Reservoir to Increase Water Supply of Panama Canal. Eng. News Rec., 107: 162–4, July 30, 1931. Illustrated description of proposed reservoir on Chagres River to augment water supply of Panama Canal, authorized by Congress in 1928. During 8 months of year, much excess water enters Gatun Lake, present source of supply, and has to be spilled into sea; during remainder of year flow is wholly inadequate. Consulting board recommended a straight gravity-type concrete dam, flanked on south end by an earth, gravel, and rock-fill structure with concrete-paved upstream face. Concrete portion will be about 900 feet long. Maximum height to roadway will be about 220 feet. Thirteen smaller dams of earth, gravel, and rock-fill construction will be required to close side valley around reservoir, which will have capacity of 22,056,000,000 cubic feet.—*R. E. Thompson.*

Notes on Cementing Constituents of Boiler Scale, Especially Silicates. F. Hundeshagen. Chemiker-Zeitung, 56: 53 and 55, 521–24 and 542–44, July 1932. As cementing constituents of scale, gypsum and anhydrite are of importance, but not the hemihydrate, presence of which is doubtful. Caustic alkalinity may occur after faulty treatment and precipitate calcium hydroxide, also a cementing agent. Amorphous silicic acid of low hydration has been observed to deposit from soft water, forming a light scale 0.75 mm. thick. In autoclave tests at 213–284 pounds per square inch, a silicic acid sol containing 5 percent SiO_2 proved to be corrosive. Author has observed as scale components an amorphous and, in absence of $Ca(OH)_2$, easily hydrolyzable monocalcium silicate and a hydrated magnesium silicate of composition $MgO \cdot 4SiO_2 \cdot 4\text{–}5\ H_2O$, which, because of its resistance to attack by dilute acid, is easily isolated. Precipitation of silicic acid with magnesia gives better results than with lime; if sufficient magnesia is available, an almost pure magnesium silicate precipitates. Neutral salts will only precipitate that part of the silicic acid which is not present as neutral sodium silicate, chlorides being the most effective. Phosphate precipitates adsorb relatively little SiO_2.—*Manz.*

Theoretical and Practical Considerations Concerning Hot Water Systems. L. W. Haase. Gesundheits-Ingenieur, 55: 12, 133, March 1932. For corrosion in hot water systems, chemical composition of cold water is of subordinate importance; overruling factors are oxygen content and nature of installation. In closed hot water plants, it is best to use a non-corroding material, i.e., copper, or cupriferous alloy, for boiler and piping. When iron is used and water is of suitable composition, corrosion can be kept in check by absorption of oxygen. In open hot water plants with air vent, and in central heating systems effective protection of boiler and expansion tank is only obtainable by constructing them of copper. Paints and various water treatments afford temporary protection; they delay the destruction, but do not prevent it. In closed plants in which both copper and iron are used, destruction of the less noble material proceeds so long as any oxygen remains. In addition to the attack by oxygen, water alone has a distinct chemical action on iron, which has to be considered.—*Manz.*

Photo-Electric Method for the Determination of Suspended Solids in Flowing Water. Paul Jakuschoff. Wasserkraft und Wasserwirtschaft, 27: 13, 152, July 1932. An isolated metal plate connected with the cathode of a battery becomes charged by light. If there be an anode near-by the cathode will discharge. The steady photo-current between the radiated cathode and the anode can be measured: it may also be intensified by charging in advance the surface exposed to the light by means of a battery current. Intensity of photo-current is, up to 800°C, independent of temperature, and directly proportional to area of radiated surface, to intensity of light falling upon it, and to duration of the exposure. By the photo-electric method first employed by Kalitin the determination of suspended solids in flowing water can be made without sampling. Errors due to extraneous light may be avoided when measurements are made inside a tube at opposite sides of which, behind glass

windows, the source of light and the photo-cell are fixed. In portable form of instrument, light-source and photo-cell are mounted as movable tines of a two-pronged fork, distance between which can be varied at pleasure to suit varying turbidities. Turbidity is read off from a curve showing the relation between quantity of suspended solids and intensity of photo-current.—*Manz.*

Group Water Supply or Individual Water Supply. CH. J. LEHR. Der städt. Tiefbau, 23: 6, 85, June 1932. Discussion of conditions which should govern the choice between providing each community of a group with a separate ground water supply and providing a central supply to serve the whole group. In favor of group water supply may be urged lack of ground water, or of hygienically safe ground water, in part of district proposed to be served; lower rentability of separate installations; lower cost of maintenance and more skilled personnel; higher pipe line pressure obtainable at lower cost and guaranteeing better fire protection. The total cost will be dependent upon length of piping necessary, location of installation, and cost of reservoir, or water tower. Examples are given to illustrate most economical installation under different conditions.—*Manz.*

Chemical Examination of the River Spree in Berlin during November and December 1931. GEORG EBELING. Gesundheits-Ingenieur, 55: 16, 181, April 1932. Investigation was made to determine influence of wastes introduced into River Spree upon quality of river water in winter within city limits. In pH value, a small decrease was observed, due to carbon dioxide from decomposition of organic matter. Minor increases noted in chloride, sulfate, and iron contents do not indicate any considerable contamination; moreover oxygen content and $KMnO_4$ consumption were but little affected. Contamination of Spree water in winter is relatively slight: of greater importance is the increase in phenolic bodies, which is considerable in the eastern industrial district, the destruction of these bodies being delayed through the low water temperatures in winter.—*Manz.*

A New Sampling Device for Bacteriological Examination of Water. EDMUND STRÖSZNER. Zentralblatt für Bakteriologie, Parasitenkunde, und Infektionskrankheiten, 125: 3–4, 250, August 1932. Device described, designed for collecting samples from wells, consists of light metal frame for holding the sterilized sample-bottle. When submerged to desired depth, glass stopper of sample bottle is lifted by a float and replaced again when bottle is drawn upwards.—*Manz.*

Ground Water under the Prussian Law and Judgments Handed down Thereunder. HERRMANN. Mitteilungen des Deutschen Wasserwirtschafts- und Wasserkraftverbandes, Berlin 1931, 32 pp. Résumé for the layman, explaining idea and nature of ground water, ownership of ground water, lawful use of ground water, legal rights of owner and restrictions to which ownership is subject, in order to protect ground water level, to protect wells belonging to others and water level of rivers, lakes, etc., and to protect ground water itself against pollution.—*Manz.*

Ground Water Rights in Switzerland. WETTSTEIN und HUG. Schw. Wasserwirtschaftsverband; Verbandsschrift 1931, 88 pp. Discussion by several members as to legal rights under which large subterraneous flow in gravel layers of the wide valleys may be utilized.—*Manz.*

Note on Velocity of Flow of Ground Water. H. SCUPIN. Zeitschrift für praktische Geologie, 39: 173, 1931. In one case, the amazing velocity of 52.48 feet per hour with an average fall of 11 percent was observed in clayey sand. From local examinations it seems probable that what was originally a flood channel had gradually filled up until no longer perceptible at surface.—*Manz.*

From Whence Comes the Karlsbad Thermal Spring O. MICHLER. Mitteilungen des Vereins der Naturfreunde in Reichenberg, 54: 3, 1932. It is shown first from their chemical composition that thermal springs of Karlsbad do not originate in the granite from which they spring, but in basalt. The tertiary upheaval at the southern border of the Erzgebirge was accompanied by intense volcanic activity; the whole district has been riven by disturbances which are of importance for understanding the thermal springs. Carbon dioxide is deemed to be the only juvenile constituent, all the rest being derived from the basalt. The water is vadose, catchment area being the southern slope of the Erzgebirge, to granite of which may be traced its radioactive content. The water sinks through fissured rock until at depth it comes in contact with basalt at high temperature and is there mineralized. Its final rise to the surface is also accounted for on basis of local conditions.—*Manz.*

On the Condition of Silicic Acid in Mineral Waters. L. FRESENIUS. Archives of medical hydrology, 10: 3, 81, August 1932. In mineral water samles with relatively high silica content examined by the author, nearly all the silica dialyzed through a collodion membrane; only a small fraction, less than 1 percent, being retained in residue. In most mineral waters, silicic acid is molecularly dispersed and, according to recent work, in form of disilicic acid; only a small part exists as colloidal hydrosol. Colloidal silicic acid is of little therapeutic effect because it readily coagulates and is only of slight migratory aptitude in the tissues.—*Manz.*

The Determination of Nitrate in Drinking Water. K. SCHERINGA. Water Pollution Research, Summary of Current Literature, 4: 7, 233, July, 1931. An amount of water containing about 0.1 mg. of nitrate is treated with 1 cc. of 0.1 percent sodium salicylate solution and evaporated to dryness on a water bath. 1 cc. of concentrated sulphuric acid is then added and the mixture allowed to stand for 10 mins., washed with water into a colorimeter glass, treated with 10 cc. of a solution of sodium hydroxide or of ammonia and compared with a standard. If the chloride content of the water is high the result is corrected by the facter 100 (100–10a) in which a = mg. Cl. Sodium salicylate is preferable to phenol owing to its greater stability.—*A. W. Blohm* (*Courtesy U. S. P. H. Eng. Abst.*).

The Effect of the Dilution Water on the Biochemical Oxygen Demand Determination. H. HEUKELEKIAN AND N. S. CHAMBERLIN. Sewage Works Journal 3: 2, 187, April, 1931. Experiments were conducted to determine what kind of dilution water is most satisfactory in making biochemical oxygen demand determinations. Three streams in New Jersey with different chemical characteristics were selected in order to show: (1) Whether natural differences in the streams within a state cause appreciable differences in B. O. D. results, and (2) which of the artificial waters give results closest to those obtained with natural waters. In addition to the stream waters selected comparisons were also made with five other prepared waters which have been used in different laboratories. The results obtained with the different dilution waters are shown in tables and discussed at some length. It is stated that the important factors which determine the value of a dilution water are: (1) the pH and buffering values; (2) the variety and concentration of ions for maintaining the necessary osmotic pressure and furnishing the necessary nutrition for bacteria.—*A. W. Blohm (Courtesy U. S. P. H. Eng. Abst.).*

Preliminary Report on the Ground-water Supply of Mimbres Valley, New Mexico. W. N. WHITE. U. S. Geol. Survey, Water-Supply Paper 637-B (1931), pp. 11, 69–90, pl. 1. Experiment Station Record, U. S. Dept. of Agriculture, 65: 6, 574, October, 1931. The results indicate that the present irrigators in the Mimbres Valley are reasonably secure in their water supply provided no large additional pumping developments are made. The quantity of water stored in the underground reservoir is very large, but the annual recharge of this reservoir is relatively small. It is considered unwise to formulate plans at this time for any large additional pumping development.—*A. W. Blohm (Courtesy U. S. P. H. Eng. Abst.).*

Bentonite. Bull. No. 107 of the Silica Products Co., Kansas City, Missouri, 1930. Water and Water Engineering, 33: 394, 502, September 21, 1931. Bentonite is defined as a natural hydrous silicate of alumina having the distinctive property of forming a highly viscous solution, sol or gel in the presence of not less than 10 times its weight of water. In the pamphlet, its composition is discussed and compared with that of other natural silicates of alumina. Physical characteristics of bentonite, of which the principal is the extremely small size of its particles, are mentioned and a method for its identification described. It is stated that true bentonite occurs almost exclusively in Wyoming, though there are smaller deposits and deposits of inferior quality in other places. Practically all of the large commercial deposits of dependable quality exist in the upper cretaceous beds, and it is suggested that it is produced by the decomposition of lava or related igneous matter through the action of water. A list is given of a large number of industrial uses for bentonite, including water softening, coagulating, clarifying and filtering; and methods of production are described.—*A. W. Blohm (Courtesy U. S. P. H. Eng. Abst.).*

A Water-borne Typhoid Fever Outbreak. FREDERICK W. SEARS. American Journal of Public Health and The Nations' Health, 21: 9, 1019, September, 1931. The outbreak occurred in Seneca Falls, N. Y., a village of 7000 people,

during August and September 1920 and resulted in 100 cases with 5 deaths. The water supply is pumped from Cayuga Lake through pressure filters directly to the distribution system. The outbreak had been preceded by outbreaks of diarrhoea with a few cases of typhoid in January 1918 and March 1920, apparently due to unusual pollution of the source of supply. A chlorinator purchased soon after the 1918 outbreak was not put into service until after the March 1920 outbreak. The outbreak herein reported was traced to contamination of the public supply from a dual connection with an industrial supply, pumped from the Seneca River section of the Barge Canal, which was intended to be used only for flushing and cooling purposes. The two supplies were connected at two points, the connection being protected by single check valves. One was found leaking and it was estimated that the leak could pass 7000 gallons per day.—*A. W. Blohm (Courtesy U. S. P. H. Eng. Abst.)*.

Purification of Water Supplies. Richard G. Tyler. Western Construction News, 6: 13, 354, July 10, 1931. There is a description of characteristics of water supplies in the Pacific Northwest west and east of the Cascades. It is stated that municipalities should have more data on temperature, taste, odors, and chemical and biological content of their water supplies over a long period of time. More experimental work should be done on the use of activated carbons for the removal of taste and odors.—*A. W. Blohm (Courtesy U. S. P. H. Eng. Abst.)*.

Detroit's Swimming Pools During 1930. C. E. Buck. City of Detroit, Department of Health Weekly Health Review, Series 13: 9, 1, March 7, 1931. A yearly summary of control results with Detroit's 55 pools. Of the total of 6144 samples, 72 percent showed zero bacterial counts and 96.4 percent bacterial counts of 200 or less with 98.6 percent showing no B. coli organisms. Seven pools have operated from 23 to 52 months without a sample showing B. coli.—*A. W. Blohm (Courtesy U. S. P. H. Eng. Abst.)*.

Progress Report of the State Committee on Water Pollution. Summarized report concerning stream pollution control and activities in Wisconsin from July 1, 1925, to January 1, 1931. The various types of major industries in Wisconsin showing numbers of each are given. Considerable research on various types of waste treatment has been carried out, mainly canning factory waste, milk plant wastes and pulp and paper mill wastes. A discussion under these various headings is given. To prevent objectionable stream pollution, the paper industry in Wisconsin has spent in excess of $1,148,000 for improved equipment and $139,000 for research. A tabulation summarizing the investigations made in carrying out the stream pollution control program is given. This summary shows an increase in industrial waste situations handled from 176 in 1927, to 311 in 1931.—*A. W. Blohm (Courtesy U. S. P. H. Eng. Abst.)*.

Drought in Ohio Valley and Water Supply. W. C. Devereaux. Monthly Weather Review, 58: 10, 401, October, 1930. This article is a brief general discussion of the drought of 1930 prepared in November, 1930, before rainfall

had again reached normal. Not only the smaller rivers were dry all summer, but many of the larger ones also, where artificial pools were not maintained. Thus the Kentucky River above Beattyville, drainage area 1,654 square miles, had an estimated discharge of from 2 to 3 second feet. Municipal water supplies had given out in practically all cities and towns where the source was unimproved rivers, wells or springs. Canalization of the Ohio River by the Federal Government has insured adequate supplies in the river cities during the driest season of record in the valley.—*A. W. Blohm* (*Courtesy U. S. P. H. Eng. Abst.*).

Future Tasks of the Engineer in Public Health. ABEL WOLMAN. Municipal Sanitation, 1: 5, 253, May, 1930. The author points out the record of the engineer in the past in the control of typhoid fever. In the future, the engineer must attack other outstanding causes of death. That this task is worth pursuing, however, rests upon the fact that the future in public health depends not only upon the prevention of disease, in itself an accident, but in the maintenance and promotion of positive health. The author's slogan for the future is "One must not only live, but one must live well."—*A. W. Blohm* (*Courtesy U. S. P. H. Eng. Abst.*).

Massachusetts—The Cradle of Public Health Engineering. HARRISON P. EDDY. Sewage Works Journal, 2: 3, 394, July, 1930. This article cites the part played by Massachusetts in the establishing of fundamental principles and the early development of public health engineering. Massachusetts took the lead in America in many factors, important among which are: (1) Boston constructed the first water works in 1652; (2) the establishment of the first Board of Health in 1869; (3) the inland waters pollution prevention act in 1886; (4) the establishment of the Lawrence Experiment Station in 1887 with its resultant development of biological principles of sewage treatment and water filtration; (5) the beginning of systematic examination of water supplies in 1887; (6) the creation of the first municipal laboratory for biological water analysis in 1889; etc.—*A. W. Blohm* (*Courtesy U. S. P. H. Eng. Abst.*).

Substances Producing Taste in Chlorinated Water. Part 1. B. A. ADAMS. Water and Water Engineering, 33: 387, 109, March 20, 1931. An excellent review and discussion of present knowledge regarding the cause of unpleasant tastes in chlorinated water, with particular reference to the so-called "iodoform" taste. It has been observed that women are more sensitive to this taste than are men and can usually detect one-fifth to one-tenth the minimum amount observed by the latter. The taste limit is far below that of odor: about one-hundredth of the amount detectable by odor can be distinguished by taste. The following ascertained and suggested sources of the phenol and other compounds causing the taste are discussed: industrial wastes, particularly gas and coke works wastes; washings from tarred roads; air-borne contamination; creosoted wood pipes; tarred cord used in pipe joints; coatings on water mains; oil on valve spindles; débris from willows, poplars and the waterside plant meadowsweet; algae, fungi and higher bacteria; decomposing organic matter; contact with metal; and iodides. The following groups of

compounds have been associated with such tastes: phonol, cresols, xylenols, salicyl derivatives, cresotinic acids and iodides. A table is given showing the lowest concentration of the various compounds usually giving taste. Substances which have been experimented with and found not to give rise to these tastes are also listed. Under specific conditions, such as absence of light, low chlorine dosage, and short time of contact, taste has been produced by author with concentrations of phenol as low as 1 part in 100,000 million. The taste produced by O-hydroxybenzyl alcohol in a concentration of 1 in 1,000 million appeared to be stronger than that given by a similar concentration of phenol. Salicylaldehyde is probably the strongest taste producer. The author's summary includes the following: (1) Taste may be due to a number of organic substances, confined to the lower monohydric phenols of the benzine series. Their mono-carboxylic, aldehydic, alcoholic derivatives, or their inorganic salts; (2) Tastes more nearly resembling those met with in practice are produced by phenol, monohydric phenols with substituted groups in the ortho position, or inorganic iodides. Para compounds yield tastes less resembling iodoform, but of greater pungency than the tastes of meta compounds; (3) The strongest tastes are produced by phenol, O-cresol, salicylaldehyde and O-hydroxybenzyl alcohol; (4) The tastable products are probably mixtures of O-chlorophenol, p-chlorophenol, trichlorophenol, or their homologues. As O-chlorophenol possesses the strongest taste and more nearly resembles iodoform, it may be said that the substances producing more intense tastes yield a greater proportion of O-chlorophenol. A bibliography of 28 references is appended.—*A. W. Blohm (Courtesy U. S. P. H. Eng. Abst.).*

American Waterworks Practice. Progress in Water Treatment. W. Gordon Carey. Water and Water Engineering, **32**: 383, 521, November 20, 1930. Brief outline of progress in water purification. Developments during the past 25–30 years have been mainly variations of processes established years ago. The artificially constructed sand filter was in use in England in 1829, the rapid sand filter with coagulate in 1885, and softening by present day processes as far back as 1888; while chlorine compounds were used to disinfect water in Belgium in 1902. Over 30 years ago Sims Woodhead employed chlorine as a temporary expedient during a typhoid epidemic at Maidstone, and in 1905 Houston applied chlorine on a large scale in London. Development of chlorination during the last 10 years has been remarkable.—*A. W. Blohm (Courtesy U. S. P. H. Eng. Abst.).*

Water Supply of Greater Vancouver, B. C. E. A. Cleveland. Western Construction News, **6**: 15, 407, August 10, 1931. A short history of the water supplies of Greater Vancouver followed by a description of the Capilano Supply, Seymour Creek Supply, Copuitlam Lake Supply, and other supplies. Also a description of the Greater Vancouver Water District, including various tables giving data and characteristics of the district and the water supplies. This is followed by a discussion of proposed works.—*A. W. Blohm (Courtesy U.S. P. H. Eng. Abst.).*

Developing a Water Supply in Arid Country. W. H. KIRKBRIDE. Railway Age, 93: 13, 427–431, 1933. To augment water storage supplying 135-mile pipe-line to various points on Southern Pacific Railroad, between Carrizozo and Pastura, N. M., dam was built on Bonita Creek to impound 384 million gallons. Dam is of rock-fill type, 90 feet high at center and 440 feet long. Upstream face is paved with reinforced concrete slab. Spillway crest is 112 feet wide. Construction details are given.—*R. C. Bardwell.*

Centrifugal Pumps Raise Water 3,153 feet in One Lift. ANON. Railway Engineering and Maintenance, 29: 4, 237–238, 1933. By installing centrifugal pumps operating against pressure of 1,470 pounds per square inch, and constructing 12,000-foot pipe-line up walls of the Grand Canyon, the Atchison, Topeka and Santa Fe Railroad is now supplying water for its own and for park service facilities at Grand Canyon, Ariz., from springs located 3,153 feet below rim of canyon. The four pumps, specially designed, operate in pairs forming the equivalent of 34 stages, and deliver 85 g.p.m. against 3400 feet under automatic operation. Galvanized seamless steel tubing, 6 inches in diameter, with wall thickness from 0.28 to 0.3125 inch, was used. Owing to height and irregularity of walls, construction difficulties were many. Water is softened in zeolite plant and chlorinated after delivery.—*R. C. Bardwell.*

Railroad Water Problems Yield to Technical Advances. ANON. Ry. Age, 95: 1, 44–46, 1933. Treatment by the railroads of approximately 135 billion gallons of water per year, or about 40 percent of water consumed by locomotives, has effected notable improvement in boiler conditions and reduced fuel consumption. Improved pumping equipment and larger tanks have reduced operating costs.—*R. C. Bardwell.*

Long Discharge Lines. R. L. HOLMES and C. R. KNOWLES. Railway Engineering and Maintenance, 29: 7, 333–34, 1933. In order to protect pipe against excessive pressures, pipe-lines across undulating country should be provided with automatic air vents at summits and blow-off valves in sags.—*R. C. Bardwell.*

Varying Water Stages. J. H. DAVIDSON and R. C. BARDWELL. Railway Engineering and Maintenance, 29: 8, 378–379, 1933. Where fluctuations in stream or reservoir exceed practicable suction lift, railroads use water-tight pits, movable units on inclined tracks, or deep well type turbine pumps.—*R. C. Bardwell.*

Is the Water Station Obsolescent? ANON. Railway Engineering and Maintenance, 29: 8, 362–366, 1933. Survey of practice of 27 representative railways, comprising 130,000 miles of line in United States and Canada, indicates that developments in locomotives and demand for better quality water during past twenty years have entailed rebuilding, or complete overhauling, of majority of railway water stations during this period, involving construction of many tanks, of thousands of miles of new pipe lines, of modern pumping equipment, and of water softening plants.—*R. C. Bardwell.*

Modernizing Water Facilities. ANON. Railway Engineering and Maintenance, **29**: 7, 328–330, 1933. During last twenty years, Illinois Central Railroad has remodeled 129 steam pumping plants, installed oil engines at 55, and electric motors at 30 others. Also, 34 water treating plants have been constructed for treating 2,000,000,000 gallons annually. Of 353 stations now in active service, 114 are supplied by municipal or private water companies and 239 are operated by railroad. Modernization of equipment has resulted in appreciable saving in operating and maintenance costs.—*R. C. Bardwell.*

Moving a 40,000 Gallon Steel Tank Four Miles. ANON. Ry. Eng. & Maint., **29**: 6, 289, 1933. Canadian National Railroad moved 40,000-gallon elevated steel tank at Bankfield, Ont., a distance of four miles without disassembling from steel tower by picking tank up with two 25-ton locomotive cranes which held tank upright while move was made to new location by worktrain.—*R. C. Bardwell.*

Zeolite Water Treatment Meets with Favor on the Northern Pacific. E. M. GRIME. Railway Age, **95**: 9, 300–302, 1933. Experience with 16 zeolite water softening plants installed on this Railroad since 1926 has been that treated water is satisfactory for locomotive boiler use under favorable conditions.—*R. C. Bardwell (Courtesy Chem. Abst.).*

Cleaning Pipe Lines with Acid. R. E. COUGHLAN. Railway Engineering and Maintenance, **28**: 10, 627, 1932. Chicago and Northwestern Railroad has had considerable success in cleaning pipe lines by circulating 50 percent solution of commercial hydrochloric acid with about 5 percent of chestnut extract added as an inhibitor against pipe-line corrosion.—*R. C. Bardwell (Courtesy Chem. Abst.).*

When a Well Fails. E. M. GRIME. Railway Engineering and Maintenance, **29**: 4, 194, 1933. Yield from wells which have failed due to incrustation of well screen with calcium or magnesium carbonate may be improved by running commercial hydrochloric acid of density 1.2 through 1½-inch pipe to bottom of well and allowing it to remain for 24 hrs.—*R. C. Bardwell (Courtesy Chem. Abst.).*

Rates for Suburban Consumers. C. KELSEY MATHEWS. Water Works Engineering, **86**: 6, 234, March 22, 1933. Rates to all consumers must be reasonable, fair, and equitable, cost of service rendered being most equitable basis. Reasonableness of rates is decided by margin of profit obtained. In municipally-owned systems where fire protection is taken care of by taxation, rates assessed against dwellers within city limits should be exempt from burden of capital cost of fire protection system; but not those assessed against suburban dwellers. In case of privately-owned systems, no such difference arises. Municipally-owned utility may also, with equity, look for higher margin of profit on business done with outsiders inasmuch as city taxpayers furnish the security upon which utility is financed.—*Lewis V. Carpenter.*

Unique Method of Financing. ROBERT E. MCDONNELL. Water Works Engineering, 86: 11, 483, May 31, 1933. Lake Springfield project, now being constructed at Springfield, Ill., represents investment of $2,500,000, an amount equal to present value of city's existing water works system. It is expected to be redeemed as to one-half, out of water works department revenues, and as to the balance, from development of marginal area of lake for recreational purposes. This conception represents unique departure in water supply financing. Area flooded is approximately fifteen miles long and, at many places, over a mile wide. Supply is designed to meet needs of 300,000 people during drought period such as might be expected once in 100 years.—*Lewis V. Carpenter.*

Submerged Pipe Line. WILLIAM J. LUMBERT. Water Works Engineering, 86: 19, 930, September 20, 1933. Town of Scituate, Mass., had to lay 10-inch main across South River in highly corrosive salt marsh mud, or soil. Transite pipe with flanged ends was used, twenty-three 13-foot lengths of pipe being bolted together on shore and then skidded into place. Pipe was then anchored down with precast concrete blocks. Hydraulic test at 40 pounds pressure showed very little leakage.—*Lewis V. Carpenter.*

Water Supply of Bridgeport. D. H. HALL. Water Works Engineering, 86: 19, 922, September 20, 1933. Bridgeport and the suburban towns of Fairfield, Westport, Trumbull, Stratford, and Shelton, with population of 203,000, are furnished with water by gravity from three reservoirs with total capacity of 11,850,000,000 gallons. Regulation valves control flow from reservoirs to city. Mains range from 48-inch down. Consumption is 26 million gallons per day. Fifty per cent of water is used by industries. Plans are prepared for adding nine and one-half billion gallons to storage by 125-foot dam at Weston on the Saugatuck River.—*Lewis V. Carpenter.*

Avoiding Legal Entanglements. LEO T. PARKER. Water Works Engineering, 86: 5, 192, March 8, 1933. Municipality may acquire land by purchase, or by condemnation proceedings, for purpose of supplying water for use of its inhabitants. City may be liable in damages for entering upon another person's property for purpose of obtaining water, where it is shown that it failed to purchase or otherwise pay for the water in strict accordance with the various laws. City may only grant such appropriations for use of water of natural running stream owned by it, as do not impair prior appropriations. City may issue permits for improvements of its streams, for beneficial use of such water, but cannot without liability authorize or make improvements which will or do result in damage to other property owners situated on stream. If an indebtedness or liability is made payable solely out of a specified fund created entirely from income of water system, and is not a general obligation of the city, consent of the voters to incur the indebtedness is not required. If, in operation of municipal water works plant, income from water rentals and the like, other than taxation, exceeds cost of operation, remainder is surplus earned from operation of the plant; but if it requires part of the taxes levied and collected to meet cost of operation, then there can be no surplus earnings. A state

statute is not repealed, unless another subsequent law is passed which clearly is intended to repeal the old law, or when new law is written in such words that it cannot be enforced without automatic invalidation of old law. Where water company has appropriated, or purchased, property and begun to supply water for public use, it cannot discontinue such service without proper authorization. A water company cannot avoid liability by transferring its assets to a mutual, or holding, water company.—*Lewis V. Carpenter.*

Water Service Obligations. Leo T. Parker. Water Works Engineering, 86: 10, 414, May 17, 1933. Laws or regulations enforcing water rates insufficient to allow fair return on investment conflict directly with Fourteenth Amendment to the Constitution. Water rates may under no circumstances be prohibitive, exorbitant, or unduly burdensome. When establishing valuations for rate making purposes, entire inter-connected operating property of utility, both used and useful for convenience of public in territory served, must be considered, without regard to particular groups of consumers of local subdivisions. Where business of a utility is both interstate and intrastate, state rates for intrastate business must be determined by considering investments in state plus value of property employed outside state, but needed and used for intrastate business. Where municipality grants to corporation franchise to conduct business of furnishing water to city and its inhabitants, and contract is made to furnish water to city for fire extinguishing purposes, and to city and its inhabitants, for other purposes, company is not required to furnish water to inhabitants and industrial plants for fire extinguishing purposes without charge, but it may adopt a reasonable rate if city charter does not regulate water rates. State law is void which requires municipality, or privately owned water company, to furnish water free of charge to various public institutions. In some states laws are in effect which give municipalities and privately owned water companies lien on real estate to secure payment for water bills in arrears. In states where law of this nature is not in effect, water company cannot shut off water supply where, for instance, receiver appointed by Court refuses to pay overdue bills for water used by firm for which he is receiver. Law or regulation is valid by which municipality or privately owned water company cuts off water to consumers who fail to abide by reasonable regulations, such as payment of water bills within a specified period after bill is rendered. Consumer of water has right to continuance of service, pending adjustment of bona fide dispute as to amount due on his bill for service. On the other hand, company has right to discontinue service upon non-payment of just bills for service furnished and has, also, right to refuse further supply of water until those bills are paid. All state legislatures have enacted laws which require creditors to file suit against debtors within specified period, otherwise debt becomes outlawed and cannot be collected. This rule is applicable with respect to municipal contracts. Landlord is bound to comply with state and city laws with respect to supplying water to tenants.—*Lewis V. Carpenter.*

The Treatment of Water by Certain Forms of Silver. J. Gibbard. Am. Jour. Public Health, 23: 910, September, 1933. This process depends upon

so-called oligodynamic action. Silver, with or without certain "activators," is deposited on the surface of sand, porcelain, or filter candles, and water is passed through, or allowed to remain in contact with, the silvered surface for some time. As a result of tests on proprietary filters, it was found that although this method of treatment has some points of interest, it cannot yet be recommended for practical use.—*H. E. Babbitt.*

The Problem of Corrosion of Well Screens. H. O. WILLIAMS. Johnson National Drillers Journal, 5: 3, 1, June-July, 1933. Comprehensive explanation of theories and principles of direct, selective, and electrolytic corrosion of metals.—*H. E. Babbitt.*

Reducing the Flow of Artesian Wells. H. E. SIMPSON. Johnson National Drillers Journal, 5: 3, 5, June-July, 1933. A superabundance of water overtaxes drainage, damages highways, etc. Valves, reducers, and plugs are used for controlling flow. Methods of placing these devices are explained.—*H. E. Babbitt.*

Excluding Salt Water from Island Wells. J. O. RIDDEL. Civil Engineering, 3: 383, July, 1933. Ground water, which is replenished by copious rains, is the normal source of supply. Because of sponge-like structure of underlying strata, overpumping causes infiltration of sea water and turns the ground water brackish. Physical principles controlling the movements of water are explained and practical methods laid down for locating and operating wells so as to avoid this difficulty.—*H. E. Babbitt.*

A Small Inexpensive Iron Removal Plant. E. S. FLANNERY. Water Works and Sewerage, 80: 4, 34, May 1933. Five parts per million of iron are successfully removed from public supply in following manner. Settling basin with two-hour detention period is equipped on top with shallow catch basin which receives the raw water and again discharges it through four spray nozzles. Water after leaving settling basin is filtered through 30 inches of sand and 24 inches of graded gravel. Entire plant was constructed by home labor at cost of $1650.—*C. C. Ruchhoft.*

Depression Water Works Engineering at Liberty, N. Y. Anon. Public Works, 64: 4, 11–12, 1933. Water supply of this summer resort village became dangerously low during summer of 1931. Studies of metered consumption indicated gross leakage, or obstruction, in water line. By means of pitot tube measurements on distribution system, important leaks were found and eliminated. Air relief valves were installed on more important high points of 8-inch delivery line and lake intake was improved. Savings thus effected in pumping costs have in two years almost equalled total amount spent.—*C. C. Ruchhoft.*

Turbidity Determinations. JOHN R. BAYLIS. Water Works & Sewerage, 80: 4, 125–8, 1933. Following instruments are described and complete directions for their operation are given: JACKSON candle turbidimeter; BAYLIS

turbidimeter; St. Louis turbidimeter, and PATTERSON turbidimeter. Last is an English instrument requiring no comparison standards for turbidity readings.—*C. C. Ruchhoft (Courtesy Chem. Abst.).*

Clarification of Water Supplies by Filtration through Anthracite. HOMER G. TURNER and G. S. SCOTT. Water Works and Sewerage, **80**: 4, 135–6, 1933. Apparatus was devised to compare respective filtration efficiencies of sand and anthracite under identical conditions. Turbid waters, prepared from red plastic clay, infusorial earth, garden soil, and clay, were applied to filters at regulated rate of 120 gallons per hour per square foot of filter area. Results indicate that anthracite is superior to sand for turbidity reduction. Superiority is believed to be due to angular shape of particles of former medium. Further experiments will be concerned with a comparison of the two media using different sized particles.—*C. C. Ruchhoft.* (*Courtesy Chem. Abst.*).

The Ammonia Chlorine Process at Richmond. MARSDEN C. SMITH. Water Works and Sewerage, **80**: 5, 157–8, 1933. Application of 6 to 8 pounds per m.g. of ammonia and 12 to 15 pounds of chlorine to raw water resulted in cleaning filters of their organic load and increasing filter runs more than 71 percent. Persistence of resulting residual chlorine caused serious tastes and odors in dead ends of distribution system. This trouble was eliminated by prechlorinating sufficiently to sterilize filter influent and adding 3 pounds anhydrous ammonia and 1 pound chlorine to filter effluent, thus maintaining residual of 0.4 p.p.m. throughout distribution system.—*C. C. Ruchhoft* (*Courtesy Chem. Abst.*).

Analysis of Water. Paper Trade Jour., **97**: 18, 36–39, November 2, 1933. Methods of water analysis adopted by Tappi Non-Fibrous Materials Testing Committee as tentative standard of Technical Association of the Pulp and Paper Industry. Exhaustive discussion of methods for determination of free carbon dioxide, color, suspended matter, residue on evaporation, organic and volatile matter, sulphated residue, silica, iron oxide and alumina, iron oxide, alumina, lime, magnesia, alkalies, manganese, sulphur tri-oxide, and chlorine. Discussion of calculation of results. Sample form of report of laboratory technicians. List of factors useful in figuring chemical combinations. Several special determinations for oil and hardness, with discussion of normal causes of hardness and relation between true hardness and soap hardness. Methods listed are devised especially for paper trade use and are supplementary to methods of A. P. H. A., and of Assoc. Official Agric. Chemists. Valuable study for the industrial chemist not familiar with water analysis.—*E. B. Besselievre.*

Water Purification and Control. FRANK R. FILZ. Paper Trade Jour., **97**: 16, 29–33, October 19, 1933. Respective merits of control by "bottle test" and by pH are compared. Optimum alum dosage has been found to be linear function of total solids and power function of alkalinity. Applicability of these relations for purpose of control is discussed. Characteristics of raw waters and purification processes of chlorination, alum coagulation, sedi-

mentation, and filtration are described in detail, including apparatus used and rates of application. In existing 2.275-m.g.d. plant, water from sedimentation basin 200 feet long and 36 feet wide with round end baffles passes through 7 Hyatt gravity sand filters 10 feet in diameter at rate of 2.87 gallons per square foot per minute. Each filter is washed once every 8 hours. Charts of operation over past 4 years demonstrate effect of rainfall on alum demand. Mathematical relations are depicted between total solids and alum requirements, and between alkalinity and alum requirements. Table is given showing agreement between theory and practice.—*E. B. Besselievre.*

Experimental Studies of Water Purification. VI. General Summary and Conclusions. H. W. STREETER. Public Health Reports, **48**: 15, 377–400, April 14, 1933. Results obtained appear to be fairly representative for waters of same general characteristics. Average efficiencies of groups of plants have shown fair degree of mutual consistency, even though raw waters were different. Diversities in performance between individual plants were often traced to variations in plant design. Bacterial removal appears to follow law similar to law of adsorption. Any bacterial limit prescribed for filtered water will therefore impose corresponding permissible bacterial limit for raw water. Bacterial efficiency can, however, be increased by prechlorination, longer periods of sedimentation, and improved coagulation, as by pH control, or increased coagulant dosage. Preliminary prolonged storage especially improves physical and bacterial quality of highly polluted waters. Results were not obtained which would justify conclusion that raw water of unlimited pollution can be rendered potable. Risk of enteric disturbances from toxic substances, organic or inorganic, in purified water must be reckoned with. Probable reinforcements of water purification include intensified chlorination, elaboration of preliminary treatment, natural and artificial storage, and increased treatment of sewage and certain industrial wastes. Weaker links of water purification should receive more attention.—*R. E. Noble.*

Experimental Studies of Natural Purification in Polluted Waters. VII. The Selection of a Dilution Water for Bacteriological Examinations. C. T. BUTTERFIELD. Public Health Reports, **48**: 24, 681–691, June 16, 1933. Author summarizes comparative results obtained in bacteriological examination of samples from seven widely separated locations in U. S., using five different dilution waters. (1) Phosphate and Formula C dilution waters give most consistent results. (2) It seems desirable to standardize on the more readily prepared, if less complete, phosphate water for further study. (3) For bacteriological purposes, dilution water must contain mineral salts: concentrations necessary, within range found in natural waters, do not appear to be critical, so far as survival of bacteria is concerned; but if growth without lag be desired, composition closely approximating natural environment would probably be best. (4) Upper limit for pH of dilution water for bacterial survival is set at 8.2; for growth without lag, at 7.5. (5) Distilled water and dilution waters with pH of 9.0, or over, are decidedly bactericidal. (6) Sterilization of dilution waters must necessarily precede their examination, as it may greatly alter their characteristics. This is particularly true for

tap and bicarbonate waters. (7) Glass used in containers must be such as to resist solution at autoclaving temperatures. Glass bottles of poor quality may cause marked changes in reaction and in mineral content. Article includes 10 tables.—*R. E. Noble.*

Court Decision Relating to Public Health. Liability of City for Sewage Pollution of Stream. Public Health Reports, 48: 24, 691–693, June 16, 1933. (U. S. Supreme Court; City of Harrisonville *v.* W. S. Dickey Clay Mfg. Co., 53 S. Ct. 602; decided 5.8.33). Since 1923, City of Harrisonville, Mo., had discharged its sewage-disposal effluent into small stream at point where it flowed through appellee company's pasture land. In 1928, Clay Mfg. Co. sued city in federal court, alleging injury to pasture land and seeking both damages and injunction and were awarded $4000 damages and were held entitled to injunction, but city was allowed 6 months within which to abate nuisance. City appealed and circuit court of appeals modified decree by eliminating $3500 damages. City then appealed to U. S. Supreme Court; latter took view that complete monetary redress could be afforded company by making denial of injunction conditional upon prompt payment, as compensation, of amount equal to depreciation in value of farm on account of nuisance complained of. Decree was reversed and cause remanded to district court for further proceedings to determine depreciation in value and to enter decree withholding injunction if such sum be paid within time to be fixed by that court.—*R. E. Noble.*

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